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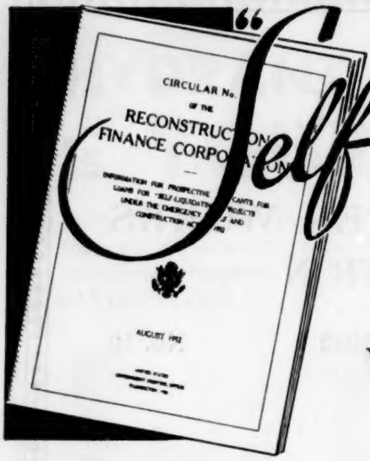

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TURBINE WELL PUMPS—THEIR HISTORY AND DEVELOPMENT[1]

By David J. Conant[2]

In preparing this paper I have kept in view the singular fact that the history of the deep well turbine pumps cannot be written by one man whose experience has been limited by service in one organization. There is no scientific record written in true academic form worthy of being called a history, but the designs and test records of more than a quarter century filed away in the archives of at least three pioneer pump builders can truly be called history. These records are essentially confidential in nature and represent painstaking efforts to overcome what seemed at that time insurmountable obstacles.

Because of the present day high state of development, disclosures of past experiences in early days by comparison appear almost foolhardy. However, the experience in the development of the deep well turbine pump paralleled and does not differ widely from that of the automobile. We are amused when we contrast the pictures of the automobile manufactured in 1905 with the present day marvels. The deep well turbine pump, unlike the automobile, is not a thing of beauty combined with remarkable performance, but rather a machine of industry which must combine efficient operation, freedom from adjustment or repair, long life and stability.

[1] From data compiled by A. O. Fabrin and David J. Conant.

[2] Layne and Bowler, Incorporated, Memphis, Tenn.

By inspection we find that the exaggerated claims for the early day pumps have now been modified or are just being realized in modern design. Lessons acquired in the hard school of experience supplemented by laboratory tests and academic research have finally built up a science of deep well turbine pump design. There has been no clearing house where all of these data could be seggregated and various concerns in their endeavors have expended much time, energy and money in private investigations. It is only logical to believe that such a procedure has incurred duplication of effort by various pump companies. Since the various designs are complex and a knowledge of these designs is considered an asset of the company, it is natural to find engineering departments, responsible for designs, more or less secretive about their work. There is still much knowledge which at present is in the process of evolution and like electricity we find the phenomena of the centrifugal pump defy explanation, but do not defy control. Through experience it has been found possible to arrive at a balance between good efficiency of operation and stability of performance.

THE DEFICIENCIES OF THE VERTICAL CENTRIFUGAL PUMP

The deep well turbine pump was developed through the shortcomings of vertical centrifugal pumps installed in dug pits. These pits were dug by hand and curbed with wood except in a few rare cases where brick or concrete were used and the pit could only be dug to a depth where the first water bearing formation was encountered and then bored wells were dug through the bottom of this pit to other water bearing strata. The vertical centrifugal pump installed close to the bottom of this pit had one or more long suction pipes entering the bore well and was driven by an exposed shaft supported from the sides of the pit and usually driven at the surface by means of a belt.

There were many dangers with such an installation, such as injury to the operator when greasing the bearing boxes of the pump shaft, accidents caused by the rotting of the curb or ladder in the pit and the presence of mine damp in a pit having poor air circulation. It was difficult to cover the top of these pits thoroughly and prevent contamination of the water by foreign material entering the pit. These plants were also limited in performance through difficulty of maintaining proper alignment of the shaft, keeping pump packing boxes tight and the fact that the pumps usually operated

on a very high suction head. With the increase in the number of wells and the demand for increased capacity from these wells it was necessary to be able to install the pump below the normal water level so that a greater draw down than the maximum suction lift could be successfully used. This condition was solved by the small diameter multi stage centrifugal pump which could be lowered into a bored well.

The early bored wells were seldom straight and were made of casing of thin wall and frequently the perforations through the casing entering the water bearing strata were cut or slashed in the casing after it had been installed in the well. This condition permitted entrance of sand with the water being pumped and led to the collapsing of many of these wells. It also caused the pump to wear quite rapidly and the success of the deep well turbine pump has been partly due to the very material improvement in bored well construction during the past several years.

Early efforts to pump water from bored wells were through the use of one of the following methods; reciprocating plunger, modified archimedies spiral or screw, air lift and multi stage centrifugal pumps. All of these types are still produced, but of these the centrifugal alone has undergone extensive improvements and developments during the last fifteen years. Improvements have been made in mechanical construction and hydraulic design which makes installation easier, repair and maintenance and the power required for pumping water much less.

THE TURBINE WELL PUMP

For the purpose of discussion, the turbine well pump can be divided into three related parts of head, discharge column and pump unit. The head which is located at the top of the well supports the weight of the pump, suspends the driving shaft on a suitable thrust bearing and provides for receiving the power either direct connected to a vertical electric motor or through belt drive from an engine. These heads effectively cover the top of the well preventing foreign material from entering the well and thus protect the water supply, and contain the mechanism for lubricating the thrust bearings and driving shaft to the pump. The heads are built in several types having the water discharge above ground or at a depth below ground suitable for connecting into water mains and in sizes from a small unit for domestic uses up to a large industrial or municipal plants of several

hundred horsepower each. In the smaller units the thrust bearing is most frequently built into the motor and is of the ball bearing type and air cooled, while in the units of larger horsepower the thrust loads are such that it is often necessary to use bearings of the Kingsbury or plate thrust which requires water cooling of the bearing chamber. With improvements in the design of the pumping units and the power shaft support it has been possible to use higher rotative speeds and the sizes of the electric motor and pump head for the same power have been considerably reduced during recent years. These modern pump heads are fully enclosed so there is no danger to the operator and are quite pleasing in appearance. A small percentage of the pump heads for use in supplying water for fire service permits normal operation through a vertical electric motor and can be driven through a belt from a gasoline engine or other separate source of power almost immediately upon failure of the main power supply.

The discharge column of these pumps connects between the pump head and the pumping unit and serve not only as a means of conducting the water from the pump to the discharge head, but also surrounding and supporting the discharge shaft. The discharge pipe is, therefore, more than a conductor for water and experience has proven that it is necessary for long life of the pump that the wall thickness must be sufficient to give rigid support of the shaft bearings and the connections between sections so constructed as to insure accurate alignment. These sections are usually of accurate uniform lengths to aid installation and insure interchangeability of parts.

The drive shaft is likewise made in sections corresponding to the discharge pipe lengths and connected by couplings usually of the threaded type and operating in bearings uniformly spaced. The bearings are either directly supported by the discharge column or at intervals through spiders to the discharge column, if the shaft and bearings are contained in the tubing which serves as an inclosure for the shaft and a conduit for the lubricant for the bearings. Pump drive shafts have been built of a variety of designs, either being along but outside of the discharge pipe or being accurately positioned within a heavy tubing inside of the discharge pipe. The bearings for the drive shaft absorb a small amount of power, but the shaft revolving in a fluid, either oil or water, its entire distance absorbs a considerable additional power. In many installations it is permissible to construct a shaft assembly within an inclosing tube and drain the tube line directly into the well just above the pump unit

and feed lubricant at the top continuously in small amounts. For this construction no packing box is required on the drive shaft and as the oil circulates through the bearings the minimum power absorption is realized. Shaft bearing materials used have been lignum vitae, babbitt, cast iron, bronze and rubber with bronze now the predominating material.

The vertical spacing of shaft bearing has received much thought from designers and constructions have varied from approximately 10 foot centers of adjacent bearings with relatively large diameter stiff shafts to small diameter quite flexible shafts with practically a continuous support for the shaft within the enclosing tube. The shaft problem is more complicated than most shaft transmissions as it is first necessary to proportion the size of the shaft and connections to transmit the required power to the pump unit and then provide sufficient stiffness in the shaft between bearing supports to prevent a critical vibratory condition being encountered within the operating speeds of the shaft. The solution of this last requirement necessitates making allowance for the tension on the shaft caused by the hydraulic thrust of the impellers which is constant throughout the length of the shaft for any fixed capacity of the pump and for the tension due to the weight of the shaft itself which varies through the entire length of shaft being greatest at the top end.

The hydraulic thrust of the impellers varies with the total pressure head on the pump at the discharge of the pump unit in the well and while the value of the thrust load can be varied considerably by details of design it must be kept as a positive down thrust on the entire operating range to prevent buckling of the shaft near the pump with consequent whipping out of shaft bearings. The initial rush of water through a pump at starting is the maximum amount of water the pump can handle and results in the least thrust load and must be provided for as this condition occurs at each start.

PERFORMANCE CHARACTERISTICS

The pump unit largely determines the performance characteristics of the installation. The overall performance of the installation is represented by the power required to drive the pump unit, the drive shaft and friction loss in water column compared to the work done in elevating the water from the pumping level of the water in the well to the point of discharge. Little has been done in recent years or now appears probable to decrease the losses in the discharge column,

excepting in the matter of using sufficiently large diameter discharge pipe to maintain relatively low velocities and to eliminate the number and extent of obstructions such as bearing spiders, tube and pipe connections or other details which cause a change in the cross sectional area of the water passage. Because of this situation the pumping unit is the logical place to expect improvement in performance to be obtained.

As in cases of other machinery we find that proper application of type plays an important part.

There are three fundamental types of centrifugal pumps. All three in operation depend in common upon the energy created by centrifugal force through the medium of a rotating impeller.

The type of centrifugal pump is determined by the specific principle used in converting the kinetic energy created by centrifugal force into pressure energy.

The three types can for the sake of convenience be called, Impulse, Reaction and Mixed Flow.

Because of the limited time available we can but touch the high spots.

Impulse type

In the impulse centrifugal pump, efficient performance is contingent upon maximum recovery of kinetic energy present in the form of velocity head at the entrance to the diffusion vanes. This velocity head at the vane ends is partially converted into pressure head while the water is passing through the diffusion vane channels.

The principles covering the design of impulse centrifugal pumps have been known for many years—in fact most European designs are of the impulse type.

The efficiency will be high for such a pump if the diffusion vane channels are formed by walls of long easy curvature having the proper area at the orifice or vane ends. However, most of the European pumps referred to have been of the horizontal type, where space was not of paramount importance. This being true, there was plenty of room to develop long and well defined diffusion channels so that recovery of energy was almost as efficiently performed as in a well designed nozzle.

The deep well centrifugal has its limits in this type because we have but little space beyond the impeller diameter to develop diffusion vanes. Due to this fact we find relatively few impulse type deep well pumps on the market.

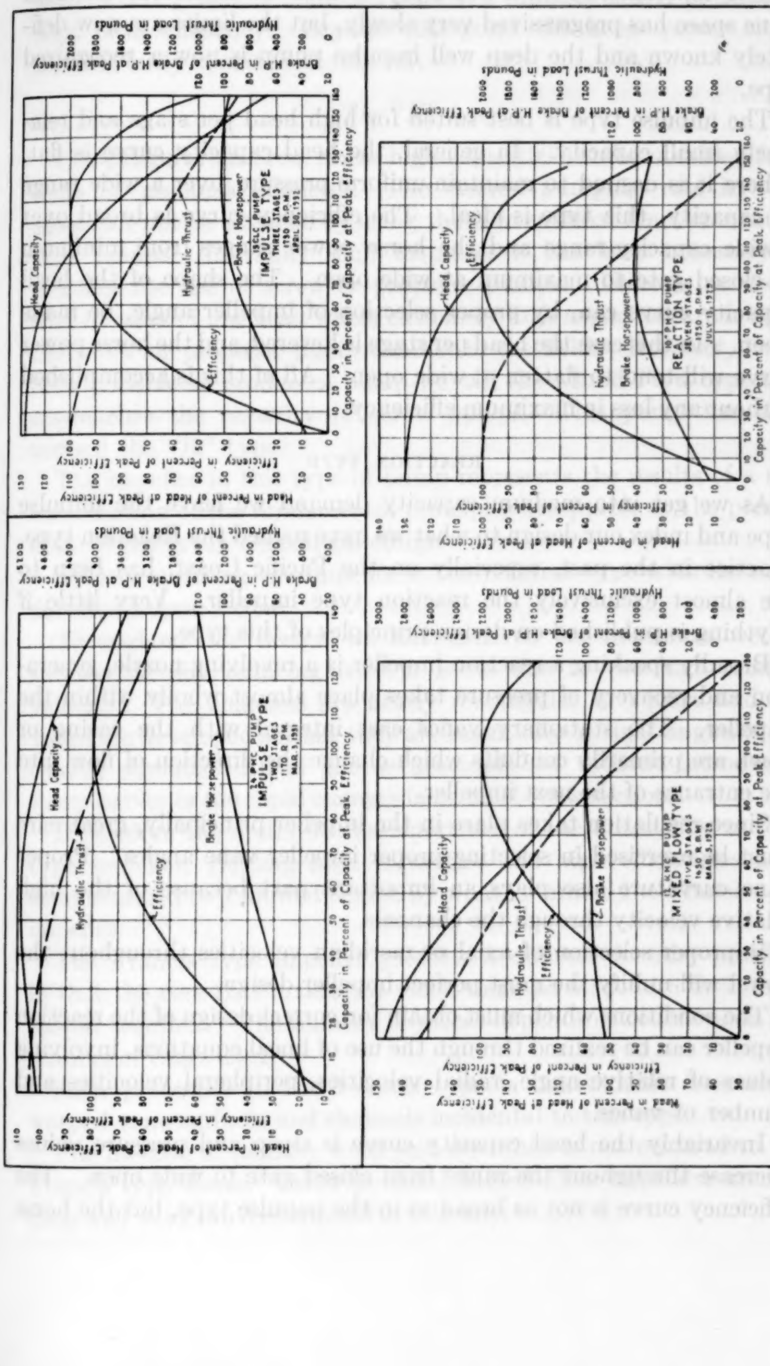
Head Capacity
Efficiency
Hydraulic Thrust
Brake Horsepower
IMPULSE TYPE
REACTION TYPE
MIXED FLOW TYPE
Capacity in Percent of Capacity at Peak Efficiency
Efficiency in Percent of Peak Efficiency
Head in Percent of Head at Peak Efficiency
Brake H.P. in Percent of Brake H.P. at Peak Efficiency
Hydraulic Thrust Load in Pounds

FIG. 1

The experimental work necessary to find the minimum diffusion vane space has progressived very slowly, but the limits are now definitely known and the deep well impulse pump is now a recognized type.

The impulse type is best suited for high head per stage and relatively small capacity. In general, the head capacity curve is flat. Where it is desired to maintain uniform pressure, over a wide range of capacity, this type is ideal. The efficiency curve is broad over a wide capacity range and the horse power ranges from minimum at closed gate to maximum at wide open. The shape of the head capacity curve can, by proper selection of impeller angle, be made steep. In this case the head per stage is lowered, and the horse power curve will tend to flatten at wide open. All of this is accomplished without any loss in maximum efficiency.

REACTION TYPE

As we get into medium capacity demand we leave the impulse type and index our design to what we have named the Reaction type. Practice in the past, especially on the Pacific Coast, has been to use almost exclusively the reaction type impeller. Very little if anything is published on design principles of this type.

Broadly speaking a reaction impeller is a revolving nozzle, generation and recovery of pressure takes place almost wholly within the impeller. The stationary vanes cast integral with the casing or bowl, are primarily conduits which change the direction of flow into the entrance of the next impeller.

Since regulation takes place in the impeller principally, great care must be exercised in selecting proper impeller vane angles. Proper vane curvature also plays an important part because of the high relative velocity through the channels.

Improper selection of axial or meridian velocities throughout the bowel will nullify the most perfect impeller design.

The conditions which must obtain for correct design of the reaction impeller can be realized through the use of lineal equations, involving values of relative angle, radial velocities, peripheral velocities and number of vanes.

Invariably the head capacity curve is steep and pressure values decrease throughout the range from closed gate to wide open. The efficiency curve is not as broad as in the impulse type, but the horse

power curve generally reaches a peak at a capacity nearly co-incident with peak efficiency. Beyond this point the horse power either remains fairly constant or falls off.

The "K" type

Conditions often arise where high capacities can be obtained from wells of nominal diameter. The desired pumping rate in such cases can not be attained by using the conventional reaction centrifugal pumps. In many such instances it has been necessary to consider the use of screw or propeller pumps. There is no doubt that in cases of low head where extremely high capacities are needed, the screw or propeller pump dominates the field. However, it has been possible to produce a pump we call the Mixed Flow type that approaches the capacity of screw pumps. This pump has been termed the "K" type.

The impeller in this type of pump represents the results of a new combination of the fundamental principles acceptable to the present state of the art in centrifugal design.

Strictly speaking the impeller is the reaction type with its vanes developed upon a cone.

Throughout the design we have striven for minimum wetted surface thereby reducing friction losses due to the high relative velocity. The channels are nearly rectangular or square and the vanes are developed so that they are essentially at right angles to the conical shrouds. All of this is accomplished without introducing sharp curves or too rapid changes in channel cross-sections.

Heretofore the desirable characteristics of the mixed flow or "K" type impeller, yielding high capacities and high efficiencies have been attained to a limited degree by what is termed the Francis type impeller.

The Francis type impeller presents a difficult foundry problem in that the core that makes the vanes must be made of 6 to 9 pieces pasted together. The number of pieces depends of course upon the number of vanes used.

Such procedure in the Francis type is unavoidable because of warped vane surfaces and channels incidental to this type.

The method of foundry work just described is not confined to the Francis type alone. Other so-called Mixed Flow impellers used in deep well work are constructed in this way.

Simplification of foundry practice was necessary to insure a consistently uniform product. With this end in view there has been devised a method of pattern construction whereby the core which forms the impeller vanes can be made in one piece.

The hydraulic performance of the "K" type indicates a steep head capacity curve, with falling pressure values as the capacity increases. The head developed per stage is not as high as the conventional reaction type, but it materially exceeds the unit pressure of the screw pump.

Unlike the screw pump whose horse power value is highest at closed gate, the "K" power curve is practically a constant value from closed gate to wide open. In wells of high capacity where levels vary between wide limits, the ideal type is the "K" pump. Minimum capacity changes through wide differences in pumping levels are possible without any danger of motor overload.

CONCLUSION

The deep well turbine pump has been built for depths as much as 825 feet and capacities as high as 10,000 g.p.m., and its limit has been nearly reached as far as shaft construction and thrust carrying capacity are concerned. It is probable that an entire departure from conventional design will need to be developed to extend the useful field of this type of pump.

Turbine well pumps can be installed to operate with high efficiency over long periods of time if the necessary operating conditions are matched to a suitable type and size of pump.

Users of turbine well pumps will profit by coöperating with the supplier of the equipment to obtain the type and size of pump unit best suited to the well conditions as to desirable characteristics of variation in total head changing the capacity and the variation in efficiency with change in head or capacity. The efficiency of the multi-stage well turbine in capacities under one thousand gallons per minute and under moderate heads can equal or exceed that available in standard horizontal pumps.

The development and availability of new metals and alloys has made possible a pump construction having high performance and long trouble-proof service. Conbinations of suitable metals in the pump construction allow economical operation in most any character of water pumped from wells.

DISCUSSION

S. M. DUNN:[3] The writer regrets that he has been unable to procure a copy of Mr. Conant's paper, and hopes that some of the following points will prove to be pertinent to the subject matter contained therein.

A discussion of the subject of maintenance of deep well turbine pumps naturally centers about the bowl section for the reason that the modern types of head and column construction leave little to be desired from the standpoint of maintenance.

Like man, the modern deep well turbine pump first feels the effects of age in parts other than the head.

Elimination of the flexible shaft coupling which, in the older coupled type of head was often of the cheapest construction, and removal of the thrust bearing from its former position in the head proper to a position above the motor have done much to make the head, as a whole, a much more reliable machine.

Improvements in design of, and materials for, ball and roller thrust bearings have also contributed to the general betterment, and in the newer designs the problem of the maintenance of the thrust bearing is greatly simplified by its more accessible and more protected location.

In the coupled type of head thrust bearing troubles and failures have been traced to vibration set up by carelessly aligned couplings which, while they compensated very nicely for angular misalignment of shafts, that were unlikely to take place, were almost useless when the shaft displacement was sideways.

Column and line shaft construction is also of a very simple and reliable sort, and the writer is not aware that any great improvements have been made or have been necessary in recent years.

The older open-type shaft with metal bearings has, happily, almost disappeared from the field, and a real source of maintenance expense is eliminated thereby. Attempts to lubricate such bearings were of doubtful worth owing to the tendency for sand to be held in the bearings by the grease, and the presence of grease in the water often rendered it almost unfit for use.

Open shafts with rubber bearings, however, have proved to be very reliable if the water does not contain an excessive amount of sand,

[3] Assistant Mechanical Engineer, Department of Water and Power, Los Angeles, Calif.

and the maintenance cost for this type of construction compares very favorably with that of the closed type. Some time is also saved in pulling and installing due to the absence of the oil column.

Troubles with closed shafts have been due principally to vibration, particularly if tension is depended upon for lateral support of the oil column. These troubles have mainly been traceable to insufficient initial tension being placed in the oil column at the time of installation. This has resulted in broken line shafts at the threaded ends or in the threaded portion of the stub shaft through the thrust bearing. Short pieces have in some cases been found to have broken off within the couplings due to this vibration, with no indication of trouble except vibration during the operation of the pump.

Pump designers have found problems in the design of the bowl section which have strained their resourcefulness to the limit, and a bowl section which is equal in durability to the horizontal centrifugal pump has not yet appeared on the market. This is probably due principally to two causes: The space limitation due to diameters of standard wells; and the presence of sand and sediment in the water to be handled.

Troubles with pump bowl sections usually take the form of worn bearings, worn impellers, and worn sealing rings. These usually occur simultaneously, but pump bowls are sometimes pulled from wells and found to be worn principally in the sealing rings.

Repairs to pump bowls usually require boring and bushing sealing surfaces, and it would seem that the cost of repairs could be decreased if bowls were fitted with more easily replaceable sealing rings both on the impellers and in the bowls proper.

In the horizontal centrifugal pump the use of various forms of labyrinth sealing rings has tended to decrease the loss of efficiency due to leakage after protracted running, but the use of rings of this design is not practicable where the water contains any considerable quantity of sand.

Tests of an 8-inch pump bowl with a capacity of 1350 g.p.m. against 50 feet of head have shown sealing ring and neck bushing leakages while new of 12 and $\frac{1}{10}$ g.p.m. respectively, and of 135 and 2 g.p.m. after 10,000 hours running time pumping water rather free of sand. At the rated capacity of this bowl section this rate of leakage would amount to a decrease of output of about 9 percent, or from the original 67 percent efficiency to 60 percent efficiency. This particular pump showed very slight wear of neck or tail bear-

ings, but an increase of sealing ring clearance from the original $\frac{1}{32}$ to $\frac{3}{32}$-inch.

In general, it is found that very few turbine pumps maintain their original efficiency for any considerable length of time, with the exception of some pumps with open-bottom runners which while they tend to maintain their efficiencies for longer periods, are apt to show marked decreases in capacities due to wear of the under side of the runner vanes.

Inter-stage bearings are found to wear very rapidly in all deep well turbine pumps, but this is not believed to be a matter of any considerable importance for the reason that except in bowl sections with large numbers of stages and consequent long, flexible shafts, it is not probable that the inter-stage bearings furnish much lateral support to the shafts.

Lubrication of tail bearings does not seem to present a very serious problem, as pumps are often found to have tail bearings in almost perfect condition after the neck and inter-stage bearings are badly worn. This wear, together with the wear which takes place in sealing rings, seems to be due entirely to passage of water charged with sand.

Except during long runs neck bearings do not get much lubrication because of the filling of the oil column with water to the standing level when the pump is stopped, and it seems that any means for preventing sand from entering with this water at the time of stoppage is valuable.

Various means have been used by different manufacturers for preventing this leakage through the neck bearing. One of the best seems to be the old reliable leather cup. Soft metal packing has been used, but is open to the objection that it becomes charged with sand, and is then worse than useless.

The writer does not believe that great improvements in efficiency are apt to be made in the future, but believes that changes in design are possible which will considerably reduce the cost of maintenance and increase the period during which the efficiency is maintained at near original value.

CONSTRUCTION AND MAINTENANCE OF DEEP WELLS IN SAND STRATA

By W. G. Lanham[1] and Thomas H. Allen[2]

This discussion of deep wells in sand strata is confined to such wells as occur in the Gulf Embayment or in geological formations of similar character.

Between the Tennessee River on the east and the Ozarks on the west, lies a huge spoon shaped bowl of bed rock with its tip just south of Cairo, Illinois and with the wide part stretching out under the Gulf of Mexico. At one time the Gulf of Mexico reached to Cairo.[2] Alternate deposits of sand and clay have been laid down on the rock to form the lower Mississippi valley. The sands and clays encountered here outcrop to the east and north of us. They go deep into the ground as we approach the Gulf of Mexico. New and more recent layers of sand, gravel and sandstone overlie these formations in what is known as the Coastal plain. In all this territory, the problem of well making is similar and involves the same problems and the same hydraulic principles, except that as the coast is approached the added problem of salt water in wells becomes a factor which we do not have in inland territory.

An approximate idea of the depth of the sands from which water is obtained is shown in figure 1.

A section through the Valley north and south in figure 2 shows in an approximate way the overlapping of the coastal formations.

Rain falling over the vast area in which these sands outcrop is the supply from which we obtain our water. It percolates through the vegetation and top soil to the sands below. It absorbs carbon dioxide from decayed vegetation and from the atmosphere. It picks up various minerals as it percolates through the sands, the most objectionable being iron. If the CO_2 and iron are removed, the water becomes very satisfactory for domestic, industrial, and boiler feed purposes.

When we consider the size of this enormous bowl, with an average

[1] Resident Engineer, Water Department, Memphis, Tenn.

[2] Consulting Engineer, Memphis, Tenn.

annual rainfall of 47 to 48 inches, it takes no great imagination to see that the supply is almost limitless. Since, however, the water has to make its way through the sands, the extraction of large quantities presents problems to the engineer seeking to provide municipal and industrial supplies in urban areas.

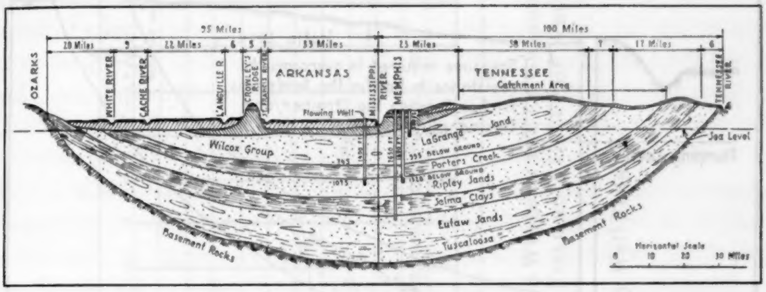

FIG. 1. EAST AND WEST SECTION THROUGH MEMPHIS, TENNESSEE, SHOWING RELATIONSHIP OF WATER BEARING SANDS AND RETAINING CLAYS

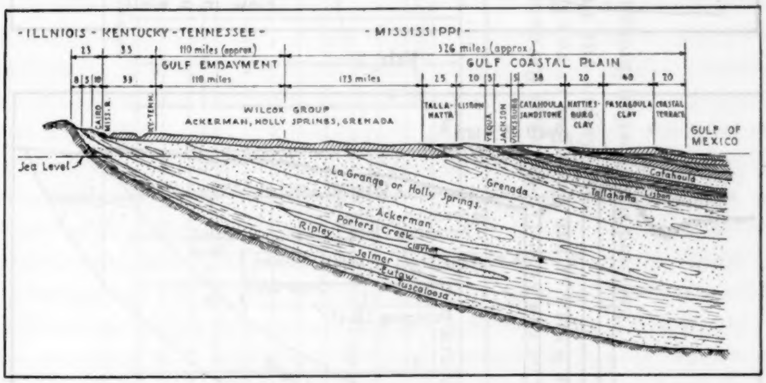

FIG. 2. NORTH AND SOUTH SECTION FROM CAIRO, ILLINOIS, TO THE GULF OF MEXICO SHOWING APPROXIMATE RELATIONSHIP OF WATER BEARING SANDS TO RETAINING CLAYS

There is a natural flow of water in these sands from north to south as indicated by a slope of several feet per mile in the original water levels.

The City of Memphis, for municipal and private supplies, is taking out of the ground an estimated total of perhaps 30 m.g.d., of which 25.5 m.g.d. are from the upper sands. This draft of water from the Lagrange sands has lowered the general level of the artesian plane and a lower drop in levels occurs at each well. Figure 3 in a simple

way accounts for the drop in water level, that occurs when a single isolated well is pumped. Figure 4 illustrates the depression that occurs in the artesian plane when a never ending concentrated draft of water occurs in an area such as exists in the well field of the Park-

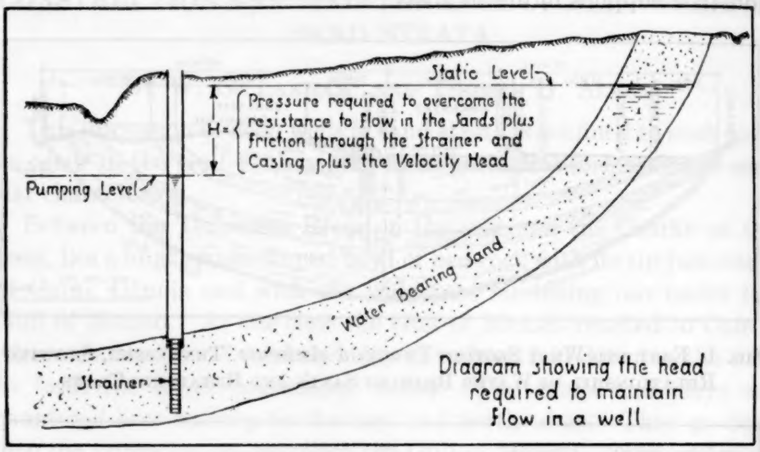

FIG. 3

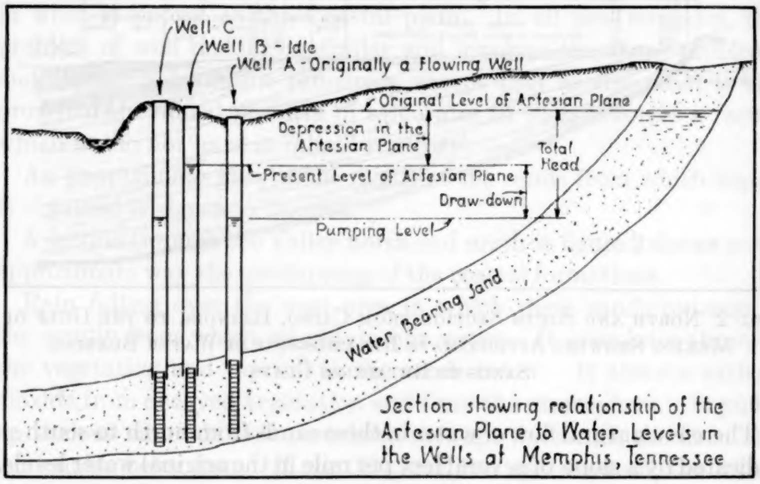

FIG. 4

way Pumping Station and to lesser extent in the area covered by the whole city. The total drop in water level is the head required to force the water through the sands, through the strainer and up the casing plus the velocity head at discharge.

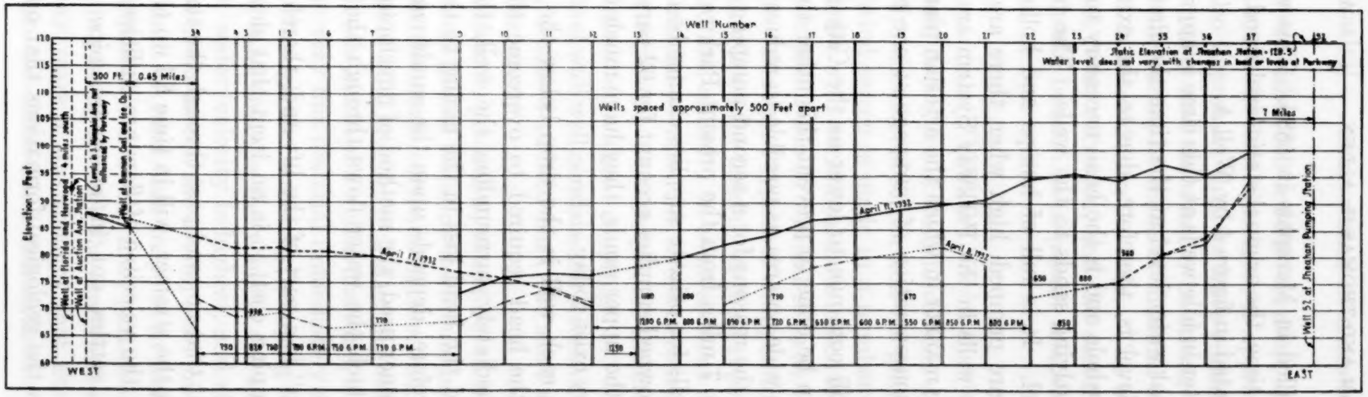

FIG. 5. VARIATIONS IN ELEVATION OF THE ARTESIAN PLANE AS RECORDED IN IDLE WELLS IN THE LA GRANGE SANDS DUE TO A SHIFT IN PUMPAGE FROM A GROUP OF WELLS AT THE WEST END OF THE PARKWAY FIELD TO A GROUP OF WELLS NEAR THE EAST END OF THE FIELD

East and West Section through Parkway Well Field

The first well was drilled in Memphis in 1886. This well was a flowing well, and, at the time, the original static level stood above the top of the well as illustrated in figure 4 by Well A.

The static level of water in idle wells at this time is approximately 33 feet below the original water level at the time the first well was drilled. This drop in level is, therefore, due to the extraction of water by the city, as a whole and is the head necessary to maintain the flow through the outlying sands to the wells. The present artesian plane is not level. It is full of humps and hollows. Low where large quantities are pumped, high when there are no wells. The levels taken in idle wells in the Parkway System are shown in figure 5 and indicate the manner in which the artesian plane may be varied in level by the concentration of pumpage at one end or the other of the field.

The artesian plane will continue to lower as the City grows and there is nothing that can be done to prevent this other than to distribute the wells over as wide an area as possible to reduce the interference. This is one of the reasons for a second pumping station in Memphis, located about 7 miles from the present Parkway Station. The concentration of wells for private supplies cannot be controlled, and since the privately owned supplies account for 62 percent of the total water taken from the upper sands, they have much to do with the artesian levels as they exist today.

The "draw-down" at each well is the drop below the depressed artesian plane, and is the head required to overcome the friction through the sands immediately surrounding the well, the friction through the strainer and the head loss in the casing to the surface. The "draw-down," therefore, depends upon the sands encountered, the type and size of strainer used, the method of developing the well and the velocities at which the water flows through the sand, the strainer and the casing.

Since the "draw-down" is a part of the lift and, therefore, enters materially into the pumping cost, it is an item that demands our attention.

It is equally important that no sand pass through the strainer into the well. Sand, passing the strainer, will in time fill up the strainer and plug the well, or if the velocites of flow are sufficient to carry the sand, it ultimately settles out in reservoirs, pipe lines and equipment.

It is to be noted from the geological formations that our wells at

Memphis are either 500 or 1400 feet deep, approximately, and further west and south the depth increases to approximately 2000 feet.

It is also important that we have reasonably straight wells, that both the casing and the strainer be strong enough to resist collapse, and that the casing be made of material strong enough to support itself as it is lowered into the hole. The well must be made of materials that resist corrosion, and the construction of the casing and strainer should be such as to permit removal of the strainer, when it fails.

TYPES OF WELLS IN REGION

In general two types of wells have been developed in this territory successfully; the slotted screen and the gravel envelope.

The slotted screen is made in a number of ways. One type is a brass cylinder with horizontal slots milled from the inside. Horizontal slots are preferable to vertical slots as there is greater resistance to collapse. A second type is a wrapped inter-locking wire soldered to vertical rods on the inside. A third type is a wire wrapped over a pipe drilled and milled. In some sections a wire gauze is used over the pipe instead of wire. A screen screwed on the end of a casing cannot be renewed. It is requisite, therefore that a wire wrapped screen be made up without couplings.

The gravel wall well consists of a screen of large openings enveloped in gravel, designed to hold back the sand and permit a free flow of water to the screen.

The problem in general is, therefore, to secure strength, to permit the maximum flow of water at the minimum loss of head, both in the well and in the sands surrounding the well, and without permitting the passage of sand into the well after the well is developed.

In the development of a well in fine sand, it is expected that a certain part of the very fine material immediately surrounding the well will be pulled through the screen by pumping at excessive rates. This removal of sand enlarges the voids in the sands adjacent to the well where the water velocity is highest, and since it permits a greater active area in the screen the overall head loss is reduced.

The amount of sand that may be so removed depends upon the velocity required to float the finer particles and the resistance they encounter in passing through the sands and screen.

For many years it was thought that the strainer slot should be smaller than the finest sand encountered. In recent years, it has

been discovered that a larger slot may be used if the fines are pumped out and that the sand pack arches the slots.

The gravel well seeks to substitute a properly sized gravel for the sands, and to thus further reduce the resistance to flow. If the voids in the gravel become in part or wholly sand packed the gain in active area is lost. If they do not to some extent sand pack the gravel permits a constant flow of sand into the well, these possibilities are the source of much debate.

A constant removal of fine sand hastens the destruction of the slotted type of screen. The rate of erosion of the slot determines the life of the strainer. The rate at which fine sand packs around the strainer and the presence of cementing materials always found in underground waters determines the life of a well of either the slotted or gravel type. In the course of future ages, the loose unconsolidated sands will be converted into sandstones. The cementing materials that will ultimately bring this about tend to close the slots of the screen. Strainers when removed often show large areas completely plugged. The slots must be designed to pass rather than to retain these cementing materials.

Straineers are usually made of brass but steel, wrought iron, stainless steel, bronze, copper, monel metal and concrete have all been used. The well casing is usually steel. In some cases, wrought iron is employed. The materials best suited are those that best resist erosion and corrosion in any particular water.

The character of the sand is the most important single factor in the yield and "draw-down." A coarse uniform sand or gravel provides the largest percentage of voids. A graded sand, a gravel mixed with sand, or sands irregular in shape provide less voids. A box full of balls all the same diameter provides the same percentage of voids regardless of the diameter of the balls. The percentage of voids determines the capacity of the sand to hold water. We speak of it as porosity.

Permeability is the capacity of the sand to transmit water under pressure and has been defined as the capacity of a sand to yield water. The permeability is determined largely by the size of the sand grains. A fine sand presents a greater wetted surface in contact with the water, restricted passageways, and due to adhesion and cohesion, reduces the yield. Temperature affects the viscosity.

The resistance to flow through the sands may be expressed mathematically in terms of velocity, the distance travelled, and the diam-

eter of the channel through which the water flows, and in form is similar to the accepted formulae on the flow of water in pipes. The difficulties of determining values for the necessary coefficients which would vary over a wide range with the variety of materials encountered makes the practical application of such a formula almost impossible.

Much has been written about the interference of one well with another. A hypothetical case can be set up to which a mathematical solution can be applied or for sand where wells have been drilled and the interference determined, we can work backwards and set up a formula, but these formulae are of little practical use.

METHODS OF DRILLING

Prior to 1890, practically all wells, exceeding 200 feet in depth, were drilled by the jetting process.

The rotary hydraulic process has practically superseded all other methods of drilling in areas where the formations are unconsolidated sands and clays, and the procedure is familiar to water works operators obtaining a supply from wells made in loose sand formations.

A hollow drill rod driving a hollow drill is rotated and lowered into the formation, while a slush composed of water and clay is forced through the rod and drill and the cuttings are washed to the surface through the hole cut by the drill. As the drill drives downward, the slush penetrates and plasters the surface of the hole and prevents caving of materials such as water bearing sands through which the hole may be driven. The slush seals the surface of the hole, preventing the water from feeding out into the strata, and enables the driller to maintain a velocity of flow sufficient to lift the cuttings to the surface. The slush that is plastered in the sides of the uncased well combined with the weight of the slush in the drilled hole almost entirely eliminates the danger of caving or breaking down of the wells while lowering the well casing into place. By the rotary process it is usually possible to set two or three thousand feet of well casing without the necessity of reducing the diameter.

It is absolutely essential that the well be so constructed as to prevent the possibility of surface contamination throughout its existence. This feature of well construction is so often entirely overlooked. Care should be taken to see that all joints are made water tight and that all damaged threads are removed from the line. To prevent seepage down the outside of the casing, an upper outer

casing is usually set with the lower end embedded in the upper clays. The well casing is then drilled through and set inside this upper casing. In gravel wall wells, an outer casing, so set as to prevent surface water from penetrating down the gravel envelope, is a necessary precaution to safeguard the well from contamination.

STRAINERS

Since the strainer or screen has such an important bearing on the useful life of a well, the type selected must depend on the conditions existent in the locality. A strainer suitable in one locality may be a misfit in some other area.

The strainer must be so designed and installed in the well as to permit removal and replacement without the necessity of pulling the well casing. Some manufacturers of wire wrapped strainers stress the ability of their strainers to resist the pulling stresses developed during withdrawal. Light weight strainers properly designed and installed can be just as successfully withdrawn, if proper methods are used.

The construction and development methods employed, the coarseness of the sand formation, the velocity through the sand and the type of pumping machinery used, all influence the width of the slot selected.

After the strainer is set, removal of the slush or mud that has penetrated the sands, becomes imperative. It is doubtful to my mind that the slush is ever entirely removed. There is no way by which we can know how far back into the sand the slush penetrates. The mud washed out may be completely removed in spots, and provide open channels through which the water passes. Strainers that have been removed show quite clearly that only portions of the surface have been working, indicating that the water finds it easier to flow around the mud remaining in the sands than to dislodge it.

The full value of the screen area may, therefore, be in part lost either by a failure to remove all the slush or by cementing of the slots. Sometimes strainers are washed down with clear water to insure that the whole surface of the strainer be at work at an even velocity of approach.

It is also, of course, quite evident that the larger the screen, the less the head loss.

It is sometimes considered permissible to use a wide slot trusting that in time the finer particles of sand will be removed and the

coarser particles will form a self-sustaining wall around the strainer. To my mind, the passage of sand is never permissible, but we do not always succeed in preventing the passage of sand.

METHODS OF MEMPHIS WATER DEPARTMENT

The drilling and development methods of the Memphis Water Department should be of interest.

The supply is taken from both the Lagrange and Ripley Formations. In the Lagrange sands, the wells vary from 350 to 550 feet in depth and in the Ripley formation, the wells are approximately 1400 feet in depth. The use of 1400 foot wells increases the supply without increasing the extent of the well field, the cost of collecting mains and plant facilities.

Since the first consideration is to safeguard the water supply from contamination, an outer casing is sunk to a depth of from 100 to 150 feet to case out the upper sand and gravel stratum. This casing is firmly driven into the impervious clay underlying the upper stratum and is left in place on completion of the well. The impervious clay averages 200 feet or more in thickness.

A test hole is drilled to discover the most desirable sand. After the well casing is set at the proper depth a seamless tubular brass strainer, having horizontal slots, is installed. The strainer is sealed against the sides of the well casing by expanding a soft lead packer attached to the top of the strainer.

The well is then pumped or swabbed to remove such clay as may have penetrated the sands and before going into service the well is pumped for awhile at excessive rates.

If contamination is found to exist, a small amount of hypochlorite of lime poured into the well and allowed to remain over night will effectually sterilize the well. One such treatment has been found sufficient.

Daily readings are made to determine the rate of flow from each well in service, and the level of the artesian plane is recorded by measuring the water level in idle wells.

The 1400 foot wells are constructed in the same manner.

As wells grow old, the yield falls off. Several measures are available for reviving the flow. For the past eight years, the following method has been used almost exclusively. The well is capped and air at 100 pounds pressure is turned into the well on top of the water. The air drives the water out through the strainer openings, which

tends to dislodge any cementing materials that may have formed on the surface of the screen and over the slots. The air pressure is then released, allowing the water to flow back to its natural level. This operation is repeated over an extended period. The well is then pumped at excessive rates of flow. This procedure, to large extent, removes the solids that have partially plugged the passageways through the sand and screen, retarding the flow of water into the well. Where air in large volumes under pressure is not obtainable, approximately the same results may be had by capping the well and pumping water into the well under pressure. This process may not be desirable with some types of strainers, but it is very effective in Memphis wells. In some instances similar results are obtained by swabbing and surging the well. When the foregoing methods cease to be effective, the strainer is removed, the sand broken down or caved, the well washed out and the strainer replaced or renewed.

For the installation of deep well centrifugal pumps, a straight hole is required. This can best be accomplished by using a sufficiently rigid drill rod, a properly shaped drill, with a proper outlet for the water. If the drill does not wander off center, the well will be straight. It is equally important that the casing be straight and properly threaded. The common output of the pipe shop is normally straight. Once the well runs off the straight line there is nothing that can be done to straighten the hole.

Seals between one size of casing and another and between the screen and the casing can best be made with a lead packer, but many devices are used. A metal to metal joint is necessary.

The hazards of drilling have been largely overcome. Drillers, engineers and water supply authorities are now thinking in terms of specific capacity, and future development will no doubt greatly increase the efficiency of ground water supply systems.

CONSTRUCTION AND MAINTENANCE OF DEEP WELLS IN SAND STRATA ON LONG ISLAND, NEW YORK

By William F. Laase[1]

Long Island lies in a generally easterly direction from the southeasterly corner of New York State, with the Atlantic Ocean along its southerly shore and Long Island Sound along its northerly shore. It has a maximum length of about 120 miles in an east and west line and a maximum width of about 24 miles, and its land area is about 1365 square miles. Fundamental rock outcrops along its northwesterly shore and dips in a generally southeasterly direction at the rate of about 60 to 80 feet per mile. Along the southerly shore the rock lies about 400 feet below the surface, at the westerly end and at a point about half way the length of the island the rock is approximately 1500 feet or more below the surface, which at this location is but a few feet above ocean tide. The greater portion of the island slopes upward from the southerly shore at the rate of approximately 20 feet per mile at the greatest width of the island, with occasional higher land rising rather abruptly to an elevation of about 300 feet above tide. Along the northerly shore of the island the land is more rugged and the upward slopes from the Sound are steeper, the highest point of land being nearly 400 feet above tide.

In general, the material overlying the fundamental rock is sand and gravel of various sizes with intervening broken layers of clays. The island, therefore, because of the open character of the soil, forms an ideal water gathering ground for a water supply system, the average annual rainfall on the island being about 43½ inches.

The City of New York has confined its water supply system to the southwesterly, westerly and northwesterly portions of the island, covering a total watershed area of approximately 250 square miles, and within this area there are many private water companies which have likewise constructed water supply works for local use both within the City of New York on Long Island and outside the City.

The term "Deep Well," no doubt, has a different meaning in

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different parts of the country. The accepted definition of a deep well on Long Island is a well that penetrates one or more clay beds. Generally speaking, these so-called deep wells show artesian conditions along or near the north and south shores of the island. The City installed its first deep wells in 1896 in the westerly portion of the watershed area referred to, and they penetrated a blue clay bed varying in thickness from about 10 to 80 feet. The water in these wells rose to a height of about 8 feet above the surface of the ground and when individually tested, with a steam driven suction pump placed directly over the well, each yielded approximately at the rate of 700 gallons per minute. These wells in general were driven to a depth of about 180 feet and were built up of 8-inch wrought iron pipe with approximately 10 feet of perforated wrought iron pipe forming the strainer, the holes being from $\frac{1}{8}$- to $\frac{3}{16}$-inch in diameter. They were rotated by horse power, and during this process, wash drilling or bucketing as required, was resorted to.

The wells in general were constructed in groups of about twelve, were then connected to a cast iron suction main and pumped under low lift from a central steam suction plant into a gravity conduit. Each group yielded approximately 3 to 5 m.g.d. The water from these wells was generally high in iron content which fact, together with the accumulations of considerable sand in the well, resulted in a materially reduced yield. After cleaning the wells, by removing the sand, it was found that the original yield could not be restored.

These plants, including all other stations on the south side of Long Island, were temporarily closed down in 1917, because of the delivery of water from the newly constructed Catskill water supply system located in New York State about 90 miles from New York City, and by 1930 and 1931, all City pumping stations on Long Island were again utilized. It was found, however, upon resumption of pumping, that the casings of these deep wells had failed by reason of corrosion which fact, together with a considerable decrease in yield, resulted in a complete abandonment of the wells and the construction of new ones with brass strainers in order to restore the original yield.

During the past two years, the City of New York has had constructed a radically different type of well as will be hereinafter noted.

WELL CONTRACT

A contract was awarded in 1930, after competitive bidding, in two sections, as follows:

Section I. The work to be done under this section consists of furnishing and delivering all materials and labor necessary for a complete installation of wells at Station No. 3 and at Flushing Pumping Station in the Borough of Queens. It includes the test wells, electrically driven pumps, motors and controls, pump houses, wiring, switch and transformer houses, instruments, gauges, meters and other electric and hydraulic devices, together with piping, valves, reserve parts, chlorinators and all accessories and appurtenances required to develop, from subsurface sources on City owned property, a supply of water of approximately 1400 gallons per minute at Station No. 3, approximately 2800 gallons per minute at Flushing Station, and a total from both the above stations of not less than 4200 nor more than 4900 gallons per minute.

Section II. The work to be done under this section consists in similarly developing a supply of water at Pumping Station No. 8 totalling not less than 2800 nor more than 3500 gallons per minute.

The Contractor's unit prices were:

Section I. $45.96 per gallon per minute of water supply developed.

Section II. $49.89 per gallon per minute of water supply developed.

The more important specification requirements are here noted as follows:

The contract required that the development shall be from the sources of water supply, that is, from wells penetrating one or more clay beds which were known to exist.

The Contractor was given the choice of numerous locations shown upon plans which were part of the contract. The minimum depth of wells was 200 feet, which was subsequently modified to include two wells, each about 100 feet deep; the maximum depth of wells was about 450 feet; approximately one-half of the wells to be about 100 and 200 feet deep.

For the 100 and 200 foot wells, the minimum outside diameter of pipe in which the pump shall be placed shall be 24 inches while that of the strainer shall be 16 inches.

For the 450 foot wells, the minimum outside diameter of the pipe in which the pump is placed shall be 18 inches and that of the strainer 12 inches.

All wells shall be of the gravel filter type.

The minimum capacity of each acceptable well was 350 g.p.m. with a draw-down of not more than 100 feet.

The well casing was to be of Armco Steel, Toncan Steel or other approved rust resisting steel or of genuine wrought iron, with a minimum thickness of not less than $\frac{3}{8}$-inch.

The well screen may be of the same metal as specified for the casing, excepting that it shall have a minimum thickness of 0.203-inch or it may be of brass construction of an approved composition.

The pump shall be a bronze fitted deep well pump of the centrifugal or other approved type of rotative principle, electrically operated and with a minimum submergence of the lowest impeller of 5 feet, regardless of seasonal changes in the water level or variation in draw-down.

The pump shall be designed for a minimum pressure of 70, and a maximum pressure of 104 pounds, depending upon location of well.

Minimum overall efficiency of pumping unit to be from 57 to 66 percent depending upon the unit yield of well.

The pump motor shall be designed for operating on 208 volts, three phase, 60 cycle, alternating current, and shall be of the squirrel cage type having a synchronous speed of not more than 1200 r.p.m. The power factor and efficiency for 100 percent of rated capacity shall be not less than that given as follows:

Horse power	75	100	150	200	250
Efficiency	88.5	90	92	92.5	92.5
Power factor	90	90	92	92	92

Sizes of the discharge piping to be as follows:

Well capacity g.p.m.	*Minimum diameter inches*
350–880	12
881–1,560	16
1,561 and up	20

A combined regulator and check valve to be installed in the discharge line from the well in order to prevent a back flow into the well and to prevent overpumping of the well beyond its rated capacity.

The Contractor shall provide Venturi meter tube with recording device and mercury manometer tube for each well constructed, and with each installation, pressure and draw-down gauges where specified.

The contractor shall be responsible for the clearing of well bacterially but not chemically and that the well cleared itself of sand within 10 minutes after the well is started, and that its turbidity does not exceed five on the Silica scale described in the standard procedure of the American Public Health Association.

Provision was made in the specifications for a 10 day test of each well for yield and draw-down, after reasonable equilibrium of the pumping water level in the well is established, the fixed draw down to be based upon the average of two draw-downs each taken for 30 minutes, one before and one after the 10 day test of the well for yield, and the fixed yield of the well, to be based upon observations taken on the mercury manometer with the pump operating against the minimum head specified and measured above the pump house floor with the pump operating at normal speed of 1200 r.p.m.

The test for efficiency shall be made with the unit operating at normal speed of 1200 r.p.m. against the maximum pressure specified, and when operating at the minimum pressure specified, the pumping unit shall not discharge more than 25 percent in excess of the discharge at maximum pressure. Duration of efficiency test—2 hours with observations taken at 10 minute intervals.

The contract provides for a deduction in payment for draw-down amounting to 25 cents per gallon per minute of the established yield of the well, for each foot in excess of 30 feet; for a deduction of one dollar per gallon per minute for each tenth of one per cent of efficiency below the minimum specified up to 2 percent beyond which the pumping unit shall be replaced, and for fixed allowances for friction in the discharge riser pipe. It also provides for periodical six months tests and inspections, and for a five year maintenance period, and the Contractor is given an opportunity to make repairs that are found necessary in any of the electrical or other equipment and to recondition the well if the loss in overall efficiency is more than 5 percent, if the draw-down has increased more than 20 percent, if the rated yield of the well has decreased more than 10 percent, or if the discharge shows sand after 10 minutes operation from starting. For failure to restore conditions as to efficiency, draw-down, and yield, in accordance with the specified requirements, the contractor is obligated to make payment to the City based upon the difference between the originally computed payment and a recomputed amount computed upon the re-test data.

CONSTRUCTION METHODS

The typical method of construction of these deep wells is here given as follows:

A heavy timber platform about 20 feet square with a central opening about 4 feet square was built in place, centered over the

well location. Upon this platform was erected a tubular four-leg derrick for handling the drilling and construction tools, the rotary machine and the necessary equipment and pipe for making the well. This derrick was about 60 feet high, about 20 feet square at the bottom and tapering to about 4 feet square at the top. Power for rotary machine was furnished through a chain drive connected to a steam driven engine set up on the timber platform, and a portable steam boiler set up nearby. A steam driven plunger pump was also set up on the platform and it was used for pumping a saturated clay solution through the hollow pipe drill rods and openings in the drilling bit into the core hole as the drilling progressed. A 10-inch core hole was drilled to a depth of about 450 or 530 feet, according to location, to bed rock, and as the drilling progressed, soil samples from the various strata were collected in a pail from the overflow of the core hole at the ground surface. The samples from the strata were collected from the pail after washing out the saturated clay solution. This wash method of collecting samples of subsoil gave reasonably reliable information for the making of the well and placing the screens, bearing in mind that the experienced well drillers on the job could, from the action or feel of the drill rod, detect the change in the strata encountered. The rate of progress in drilling the 10-inch hole was in general 100 feet per day.

After completing the 10-inch guide hole, the upper part for a depth of about 100 to 160 feet, depending upon the position of the clay bed, was reamed to a diameter of 32 inches. Into this 32-inch hole, 26-inch steel casing was lowered to rest on a natural clay shelf where the 32-inch reamed hole was stopped. Below this 26-inch casing, the 10-inch guide hole was reamed to a diameter of 24 inches, to a depth of about 290 feet below the surface. In this hole, 18-inch casing was lowered, the bottom resting in a clay bed just above the water bearing strata. The 18-inch casing was later cut off, leaving a lap of about 50 feet between it and the 26-inch casing, and the annular space sealed with a quick setting cement. Below this 18-inch casing, the 10-inch guide hole was reamed to a diameter of 16 inches for a depth of about 130 feet, or about 460 feet below the surface. Into this hole, a string of 12-inch screens or strainers with 12-inch pipe above same were lowered into the hole and the screens were set in the water bearing strata with intervening blank 12-inch pipe opposite one or more existing thin clay beds, the 12-inch pipe above the screen being temporarily carried to the surface. During

the process of reaming for the different sizes of pipe, the pumping of the saturated clay solution into the hole was continued, and samples of the soils encountered were taken as a check against those collected from the 10-inch core hole. In cases where boulders were encountered, progress was somewhat delayed because of the necessity of washing and moving them ahead or aside of the drilling bit.

The pipe used in the wells, including the screens which were of the shutter type, was of Armco iron, in 20 feet lengths either riveted together or butt welded end to end, excepting the 12-inch pipe which was of genuine wrought iron with screw threads and couplings, and excepting that the 12-inch screens were coupled together with screw threads and couplings.

In the annular space at the surface, between the 12-inch and outer casings, $\frac{1}{4}$-inch gravel was placed.

The work of cleaning and developing the well was performed by agitation and pumping through the use of a close fitting wooden plunger worked up and down for the full depth in the 12-inch pipe, for the removal of the clay solution which entered through the screen from the outside, water being supplied under high velocity from jets set at an angle through the plunger. By this process, much of the saturated clay solution used in the drilling of the well was removed and the sand drawn into the well was removed with a plunger sand bucket. As this work progressed, one quarter inch gravel was continually fed from the surface into the annular space to replace the sand and clay removed from the outside of the screen. This operation was continued until the wooden plunger reached the bottom of the well where a concrete plug was set to close the well. At this stage of the work, the 12-inch casing above the screen was cut off, leaving a lap of about 75 feet between it and the 18-inch casing. The well was further developed through the use of a 15-inch, 4-stage horizontal, belt-driven centrifugal test pump lowered into the well many feet below the water level, the power for operating the pump being supplied by a 100 H. P. gasoline engine. Through the intermittent operation and stopping of this pump, additional clay solution and sand was removed until the water from the well became clear in accordance with the specification requirement. The pump was then continuously operated for a period of about 10 hours in order to fix the rated yield of the well from measurements taken on a submerged circular orifice attached to the end of a large discharge pipe. From the data thus obtained, the permanent pump and all appurtenances

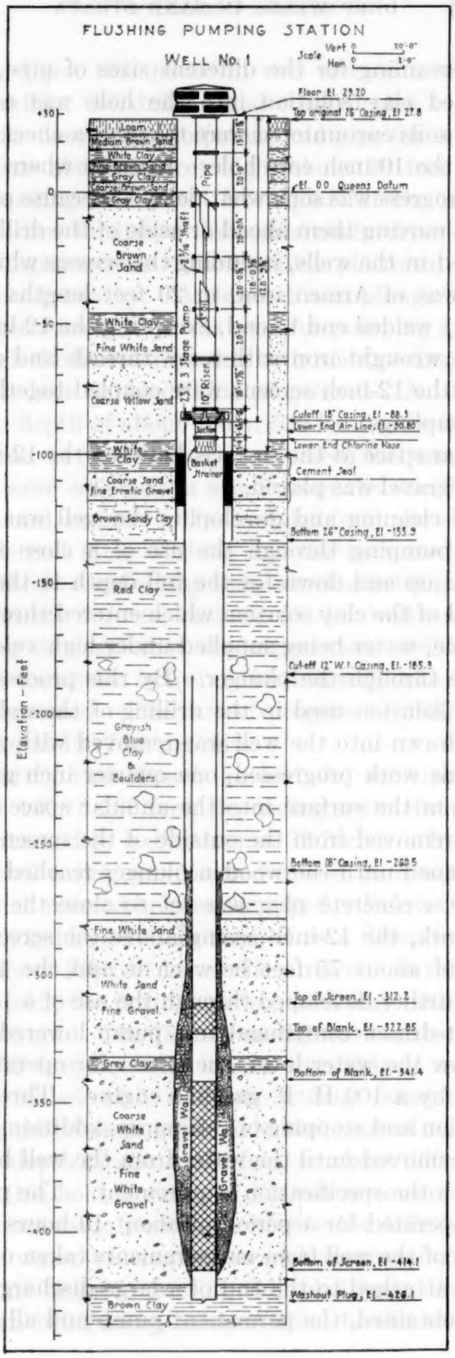
FLUSHING PUMPING STATION
WELL No. 1
Scale Vert 0 20'-0"
Hor 0 3'-0"
Floor El. 29.20
Top original 26" Casing, El 27.8
El. 0.0 Queens Datum
Cutoff 18" Casing, El -88.3
Lower End Air Line, El -90.80
Lower End Chlorine Hose
Cement Seal
Bottom 26" Casing, El -135.9
Cut-off 12" W.I. Casing, El. -185.3
Bottom 18" Casing, El -260.5
Top of Screen, El. -312.2
Top of Blank, El. -322.85
Bottom of Blank, El -341.4
Bottom of Screen, El -414.1
Washout Plug, El -426.1
Elevation - Feet
+30
0
-50
-100
-150
-200
-250
-300
-350
-400
Loam
Medium Brown Sand
White Clay
Fine Brown Sand
Gray Clay
Coarse Brown Sand
Gray Clay
Coarse Brown Sand
White Clay
Fine White Sand
Coarse Yellow Sand
White Clay
Coarse Sand
Fine Erratic Gravel
Brown Sandy Clay
Red Clay
Greyish Blue Clay & Boulders
Fine White Sand
White Sand & Fine Gravel
Grey Clay
Coarse White Sand & Fine White Gravel
Brown Clay
10" Riser Pipe
Basket Strainer
Gravel Wall

FIG. 1

TABLE 1

Summary of deep well construction and test data, Long Island, New York, April, 1932

WELL NUMBER	DEPTH OF WELL, FEET	DIAMETER OF CASING, INCHES	SHUTTER-SCREEN		DRAWDOWN		OVERALL EFFICIENCY, PER CENT			CONTRACT UNIT PRICE, DOLLARS	YIELD, G.P.M.	GROSS COST OF WELL, DOLLARS	DEDUCTIONS FOR		NET PAYMENT FOR WELL, DOLLARS	NET COST PER G.P.M., DOLLARS	WELL NUMBER
			Length, feet	Diameter, inches	In feet	Excess, feet	Actual	Required	Minus				Excess drawdown	Minus efficiency			
Section I. Station no. 3																	
1A	92	38–26	30	26	28.194	0	65.8	62.0	—	45.96	781.675	35,925.78	—	—	35,925.78	45.96	1A
2A	109	26–18	40	18	17.247	0	60.5	57.0	—	45.96	541.517	24,888.12	—	—	24,888.12	45.96	2A
Flushing station																	
1	455	26–18–12	84	12	32.777	2.777	70.4	65.0	—	45.96	1,463.285	67,252.58	1,015.89	—	66,236.69	45.266	1
5	421	26–18–12	85	12	38.600	8.600	71.4	66.0	—	45.96	1,560.000	71,697.60	3,354.00	—	68,343.60	43.810	5
Section II. Station no. 8																	
1A	215	26–18	30	18	78.391	48.391	64.2	65.0	0.8	49.89	996.942	49,737.44	12,060.76	797.55	36,879.13	36.993	1A
3A	143	26–18	35	18	66.740	36.740	66.7	65.0	—	49.89	1,144.890	57,118.56	10,515.81	—	46,602.75	40.705	3A
3	523	26–18–12	90	12	60.483	30.483	65.0	66.0	1.0	49.89	1,358.168	67,759.00	10,350.26	1,358.17	56,050.57	41.269	3
Totals											7,846.477	374,379.08	37,296.72	2,155.72	334,926.64	42.685	

were designed by the Contractor in accordance with the specification requirements, and later installed by him. The time consumed in drilling the core hole, reaming, placing the well casings and screens, averaged about 35 days for the deepest well, and the time consumed for developing the yield of the well, including the preliminary test of same, averaged 20 days.

A typical section of the wells, showing the details of construction and the strata through which the well was constructed, is shown in figure 1.

A schedule showing briefly the results of the official tests of the seven wells constructed, together with the cost of same to the City, is given in table 1.

OPERATION

Five of the wells are now in constant operation, and no defects or other serious troubles have developed. No maintenance work has as yet been found necessary, and the indications are that these wells will maintain their rated yield, draw down and efficiency as indicated by a trial test of Well No. 3 at Station No. 8 where the efficiency of the unit has been increased about 5 percent by reason of its continued operation.

The two wells at Flushing have been operated only occasionally because of the high iron content in the water (2.5 to 4.5 p.p.m.) pending the construction in the near future of iron removal filters.

The cost of operation and maintenance, including fixed charges for the seven wells, is estimated as follows:

Fixed charges

Interest on investment, 4 percent on $334,926	$13,397.04
Sinking fund, assumed 20 year life and 4 percent interest on contributions (0.03358)	11,246.81
Total fixed charges	$24,643.85

Maintenance and operation

Labor for operation:	1 engineman	@ $3,000	$3,000
	2 oilers	@ $,1500	3,000
Maintenance, repairs and supplies, 7 units @ $333.53			2,334.71
Total developed yield 11.3 × 365 = 4124.5 m.g. per year			
Average yield (75 percent load factor) = 3093.4 m.g. per year			

Cost of power @ 0.013 k.w. or \$17 per m.g., \$17 × 3093.4	52,587.80	
Total cost, maintenance and operation	\$60,922.51	\$60,922.51
Total yearly cost of 7 wells		85,566.36
Total cost per m.g. $\left(\frac{\text{Total yearly cost}}{3093.4}\right)$		27.661
Cost per foot million gallons		0.1044

THE LOSS OF WELLS IN CONSTRUCTION AND OPERATION

By D. A. Lane[1]

To those responsible for the water supply of a city, largely dependent on wells, it is of more than passing interest to determine where the greatest destruction of wells occurs.

For the purpose of determining this, it is imperative that not only a large group of wells be studied, but also that a long period of time be covered to allow all factors to enter the picture.

That some of the well losses described in this paper may be better qualified, a short historical résumé of the water supply of Los Angeles is given.

The first wells to augment the surface and gallery supply were drilled about 1897 in the Los Angeles River area. This supply was sufficient until the drouth of 1905, which determined the City on the necessity of going to the Owens Valley for an additional supply which would be independent of local variations in rainfall. To supplement the local supply until completion of the Owens River project, deep wells were drilled in the southern portion of the City. The Owens River surface supply was made adequate for the growth of the City by additional water right purchases. In 1924, the statewide drouth threatened all sources of supply and Mr. Wm. Mulholland, then Chief Engineer of Water Supply, began a systematic development of groundwater, both locally and in the Owens Valley, to be used during emergencies. This program has been carried on until this study includes 395 wells drilled during a period of over 35 years.

The total number of wells lost or abandoned during this period amounted to 29.1 percent of the total drilled or acquired by annexation.

Of the total number of wells abandoned, 25.2 percent were lost during drilling and 74.8 percent after being put in operation.

It is most difficult to determine the specific cause of failure as it is very easy to confuse cause and effect, and the assigning to various arbitrary classifications is a matter of personal judgment only.

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CAUSES OF DRILLING LOSSES

Drilling losses were divided into six classes and the percentage of each computed on the number of wells lost before being put in operation.

Casing

The predominating loss was found both in the use of casing too light for the job, and over perforation, with 20.7 percent each. The initial cost of casing of sufficient weight is small in comparison with the many difficulties directly attributable to its lack of use, both in drilling and in the life of the well.

Over perforation

Over perforation is a most common fault and found quite often in wells that should present no difficulty. The presence of fine sand with a minimum quantity of screening gravels may be safely and adequately perforated by a new type of perforator recently developed by a well known southwest driller and has been successfully used by the City of Los Angeles.

Carelessness or misjudgment on the part of the drill crew or engineer in charge is responsible for the loss of some wells. If the driller has not kept a proper log he may perforate in a material too fine to form a screen, or the crew in turning some types of perforators may cause two or more holes to be cut close together, tearing the casing. Occasionally the engineer in an attempt to procure more water may have the driller perforate in a material too fine.

Accidents

The loss of tools and the damaging of the casing by the breaking or wearing away of rivets on the inside joints was accountable for 17.2 percent of drilling loss. In the use of riveted stovepipe casing the proper alignment of the seam will eliminate a large part of this danger. Unquestionably, the stovepipe casing of the future is the welded joint. It is stronger, safer and faster to assemble as no time is lost in matching seams. It has been used successfully in the Los Angeles area up to depths of 1700 feet with 16-inch diameter, and 1000 feet with 20-inch diameter casing.

Formation and crooked holes

Obstacles encountered in the formation may result in various troubles to the well, the principal ones being a crooked hole or

telescoped casing, in either case the well must be abandoned or the casing pulled. If considered as individual causes formation and crooked holes have a percentage of 17.2 percent each, or a combined loss of 34.4 percent.

Lost in development

Wells lost during development, of course, may be the result of damage caused under some other divisions and not show up until this stage of operation. It is, therefore, impossible to assign a direct cause. The presence of the driller on the job at all times during development is a valuable asset as he is able to recognize the various formations as they develop. This is especially true of wells drilled in areas where sand strata predominate, as the overdraft of one particular stratum may cause the overlying formation to cave and ruin the well. The driller recognizing the upper stratum may shut down the well before caving, thus allowing the installation of a liner. The smallest loss occurred under this class, being only 6.9 percent.

WELLS LOST AFTER COMPLETION

Collapsed well

The majority of wells lost by collapse of the casing are those in areas where sand strata are perforated too heavily and continuously make a small amount of sand. Eventually the overlying material sloughs off, resulting in a collapsed or sealed off well. Wells lost in this manner constitute the second highest loss in completed wells with 10.4 percent.

The solution of this danger is covered under "lost in development."

Obsolete by reason of diameter

The greatest loss to the system occurred by reason of small diameter wells with 30.3 percent. With the development of efficient large capacity pumps the operation of many small units has become most uneconomical. For this reason the replacement of all wells should, as a matter of policy, call for the drilling of wells not less than 16-inch diameter.

Obsolete by reason of location

With the rapid growth of communities and the necessity for changes in sources and methods of supply, it is inevitable that over a period of years wells would be drilled in areas that would later be unprofitable to operate. This resulted in a loss of 24.4 percent.

The combined loss to the system for both obsolete location and diameter is 54.7 percent of all wells lost after completion.

Lowered water table

Notwithstanding the receding groundwater levels of Southern California, only 7.0 percent of those wells were from this cause. Of these, 50 percent were lost due to the construction of a deep cut by private parties for a gravel pit, which resulted in draining the storage basin.

Contaminated water

The only wells lost by reason of contaminated water were those of a nearby community in an isolated valley which had no sewerage system. As the pollution began to affect its wells annexation to the City of Los Angeles resulted and an outside supply was brought in. When the area had been cleared by proper sewer systems the wells proved of such small capacity that it was uneconomical to operate them. Applied to the total number of Los Angeles City wells lost, this group amounted to 7.0 percent.

Miscellaneous losses

Other losses occurred from various causes, such as lost or jammed pump, 2.3; reperforation, 3.5; casing ruined by bad water, 3.5; destroyed by dynamite, 3.4; and destroyed by floods, 2.3 percent.

Under the class of wells lost because of salt water or dry hole at the time of drilling, may be placed 5.8 percent. This is placed under the division of completed wells due to the fact that the drilling was completed to depths desired for operation.

CONCLUSION

This survey of the wells of the City of Los Angeles covers all conditions and phases of well drilling in the Southwest from sand predominating coastal wells, with depths of 800 feet below sea-level, to heavy boulders and lava in the Owens Valley over 300 miles distant. For this reason the losses compiled may be applied to the average groundwater development in this area.

A glance at the maximum losses shows that under drilling losses, light casing and overperforation were the cause of 41.4 percent of the wells abandoned, or 3.4 percent of all the wells drilled.

Under the completed class, 54.7 percent of the wells lost because

of obsolesence, or 11.9 percent of the total number of wells drilled. This was followed by collapsed wells with 10.4 percent of those abandoned, or 2.3 percent of those in the system.

While the majority of the collapsed and overperforated wells were in sand producing areas, the large percentage of losses attributable to other causes indicates the necessity for careful observation of wells at all stages of drilling and operation.

THE MEMPHIS WATER SUPPLY

By F. G. Cunningham[1] and Wellington Donaldson[1]

The entire water supply of Memphis is obtained from deep wells. The strata which furnish the supply are, in importance and extent, among the outstanding water-bearing formations in the country. They are part of a group which extends, practically without interruption, down the Mississippi Valley from Cairo to the Gulf, and along the Gulf coast and Atlantic seaboard and as far north as New York. Within this huge distance the strata, of course, vary in texture, depth and geologic designation, and no one stratum is continuous for the entire distance, but the broad statement holds.

The average water demand on the municipal system of Memphis is about 19 million gallons daily, to supply 250,000 persons. Perhaps as much more water is taken from privately-owned wells, by industries, hotels and the like. In few other cities in America does the need for so large a quantity coincide with the ability to get it from the ground. Houston, San Antonio and Camden are perhaps the only comparable cases. A much larger quantity is taken, in the aggregate, from wells on the western end of Long Island, but the total is divided into dozens of independent, scattered developments, serving many municipalities. Of the large cities using ground water supplies, Memphis is, so far as known to the writers, the only one in which all of the water is brought to one or two large central stations for treatment and pumpage into the mains. In most if not all other large supplies the wells are scattered over a great area, and the water is pumped into the mains either at each well or at a number of comparatively small stations, and without treatment.

The Memphis Water Department is now building a new set of water supply, treatment and pumping works. These are known as the Sheahan Station and constitute the second of two large central plants. These plants together, are expected to serve all needs for the next 25 years or more. The first of the two, known as the Parkway Station was completed in 1924 when it replaced all prior

[1] Of Fuller and McClintock, Engineers, New York, N. Y.

supply and pumping facilities. The plant now being built completes the major improvement program laid down when the Parkway Station was designed. Each station comprises a chain of wells with collecting piping, an aerator to further reduce carbon dioxide partially removed by air-lift pumping, mechanical filters to remove iron, a steam pumping station, and basins to store filtered water. The two stations are about 6 miles apart and will have an aggregate maximum capacity of some 58 million gallons per 24 hours. In each plant the water is raised to the ground level by air-lift, air being supplied from steam-driven compressors in the pumping station. The water collected from the wells is then raised through a moderate lift to the aerator, whence it flows by gravity through the filters and to the high-service pumps. The stations and equipment are described more fully in the latter part of this paper.

SOURCE OF SUPPLY

The main source of ground water supply in western Tennessee is a blanket of sand known as the Lagrange stratum, which at Memphis is found at a depth of 300 to 500 feet, and has a thickness of 50 to 150 feet. The stratum is fed principally by an extensive outcrop area which crosses the state in a band roughly paralleling the Mississippi River and passing some 40 miles east of Memphis. At each of the two Memphis waterworks well fields the water level in wells penetrating this stratum stands about 50 feet below the ground surface when the field is not being pumped.

About 1000 feet below the Lagrange stratum and separated from it by impervious layers are the Ripley sands. These had not been developed for water supply in Memphis, to any extent, until the water department began exploring them at the Parkway Station after its completion in 1924. Since then 9 wells in this deeper stratum have been added to the 22 Lagrange stratum wells that comprised the original Parkway development. The Ripley stratum appears to yield less freely and has a smaller outcrop area than the Lagrange sands and wells in it of course cost more for construction. However, by developing both the Ripley and Lagrange strata in the same well field, a much greater yield may be obtained in a given area and with a given lift than could be had from the Lagrange sands alone, thus saving in piping and land costs and facilitating operating control. At the new Sheahan Station about half of the wells will be in each stratum. The standing water level in wells in the deeper stratum is

somewhat higher than in those in the Lagrange stratum, as would be expected from the conditions. The difference is quite marked at the Parkway Station and is slight at the Sheahan Station, 6 miles southeast.

In both strata the sand is relatively fine, although quite variable in size. Strainers having fine slots and liberal length are required, but on account of the great thickness of the sand beds, yields of one to two million gallons daily per 10-inch well are readily obtainable, provided the wells be spaced 500 to 1000 feet apart.

There is no question as to the adequacy of the underground supply in Western Tennessee, as there so often is in connection with ground water supplies. It has been used freely at Memphis for over 40 years, without lowering the general ground water level in the City more than 30 feet. What lowering has occurred is due to the steepened hydraulic slope set up by the gradually increasing consumption, rather than to any depletion of the supply. Evidence from past use and knowledge of the outcrop area assure an ample supply for Memphis for many years to come.

HISTORICAL

The first water supply for Memphis, developed in 1872, by a private company, was from Wolf River, a small turbid stream that skirts the northern part of the city. Dissatisfaction with the water quality led to its abandonment in favor of a well supply in 1889, following the successful development of wells by a rival company. The well supply plant built then, known as the Auction Avenue plant, was for many years the sole source of supply for the City, and its use continued as the major source until it was abandoned upon the completion of the Parkway Station in 1924. It was located about 1½ miles north of the business center. The city acquired the works in 1903 by purchase from the Memphis Artesian Water Company.

The Auction Avenue works, when built were notable for certain features of design and construction. Neither air-lift wells nor centrifugal well pumps were then in commercial use, and direct pumping from wells was the only satisfactory method available for large projects. In this case some 40 scattered, bored wells were connected to the pumping station by several miles of main and branch tunnel, built in clay about 80 feet below the ground surface. The pumps at the station were set in a deep pit, to be within suction reach of the lowest level to which water from the tunnels might be

drawn. All connections between the tunnels and the wells were made in the tunnels, and various ingenious expedients for this and other purposes were employed.

Before 1921, when the writers' firm was employed, the Auction Avenue station had been supplemented by a small air-lift plant in East Memphis, and 14 independent wells, each of the latter having electrically-driven pumps delivering the water to the mains. All three groups of facilities, which were scattered over different quarters of the city, had then been outgrown. Costs for fuel, power, operating labor and maintenance were high, due to the scattered arrangement, diverse character, small size, age and, in some cases, obsolete type of the component parts of the system. There was no storage of water and all variations in demand had to be met by the well system. Also, for over thirty years the consumers had suffered from red water troubles, caused not only by the natural iron content of the water, but by the corrosive effect of the high carbon dioxide content.

In general there were in 1921 three main issues to be decided, namely:

First, whether the supply thereafter should be taken from wells or from the Mississippi River.

Second, assuming it should be decided to continue the use of ground water, what should be the arrangement and character of the new works.

Third, the feasibility and cost of treating the water to correct red water troubles.

No extensive study was required to eliminate the river supply for reasons of cost. It was estimated that the total cost would be \$200,000 per year greater than for new ground water works, and the quality of the river water, even when purified, would be inferior.

When it came to deciding upon the form and location of improvements to the well supply system, there was almost an embarrassment of alternatives open. Wells could be placed in any quarter of the city; the entire supply could have been obtained from one new station or any number of smaller ones; and parts of the existing system could have been retained. These questions were settled, after numerous studies, simply on economic grounds. It was found cheaper, and infinitely more satisfactory, to scrap all of the existing facilities except the distribution system, than to retain them. It was also found that the economies of one favorably located, large station for the present, and another for the future, would outweigh

the savings in distribution piping obtainable from the use of scattered small stations.

Experiments showed that carbon dioxide in the well water could effectively be removed by air lift pumping followed by a coke-tray aerator, and that iron could be removed by ordinary filtration. Since the costs of treatment were moderate compared with the expenditures required in any event, its incorporation in the plan was authorized and became another reason for centralizing the facilities at large plants.

Accordingly the Parkway Station, which now supplies the entire city, was built, and put in service in 1924. It is located about 2 miles northeast of the business center, and its function in the ultimate program is to serve primarily the older and more densely populated western part of the city. It was recommended at the same time, that a similar station be built about 1930, near the easterly edge of the city, since the major growth would obviously be in that direction and the extra cost would ultimately be offset by savings in distribution mains and greater reliability of service. The Sheahan Station now being built fulfills this part of the plan, and incidentally was initiated in the year originally contemplated.

Memphis therefore has gone through the experience, unusual for a large city, of discarding all of its water supply and pumping works at one stroke. In return she has secured comprehensively planned modern works that should serve, with only unit extensions, for several decades, eliminated red water troubles, greatly reduced operating costs, improved the reliability of service and secured a higher underwriter's rating. The water department meets all expenses including funded debt requirements, with its own revenues and has carried out the improvements without raising water rates. The Parkway Project fixed charges have been largely covered by operating savings.

PARKWAY STATION

The Parkway Station has a capacity, for the maximum 24 hours, of about 28 million gallons. The high service pumps can meet hourly peak rates up to 60 m.g.d. The station has been developed to its intended limit. It is supplied by 31 wells mostly located along a boulevard thoroughfare and extending east from the station for two miles. The collecting pipes from the wells and air pipes to the wells are buried in the central parking strip of the boulevard and the

wells are mainly on adjoining building lots. The pumping lift at the wells is about 100 feet, and the well tops and grades are such that the water flows by gravity to the central station. There the water is raised 25 feet to the aerator, passes through the aerator and filters by gravity, and is pumped to the mains under a head of about 175 feet.

The lift to the aerator is accomplished by centrifugal pumps driven by reaction water-turbines. The driving water is taken from the high service pumps and, after passing through the turbines, is used as cooling water for the condensers of the air compressors, and returned to the suction side of the high service pumps. The arrangement has been simple, economical and satisfactory.

The five compressors are cross-compound two-stage machines, each having a capacity of 2700 cubic feet per minute and operating normally at 90 pounds air pressure. They develop an indicated horse-power in the steam cylinders on about 10 pounds of steam per hour.

The high service pumps comprise three turbine-driven centrifugal units and two horizontal, cross-compound pumping engines, each being of 15 m.g.d. capacity.

The steam conditions are favorable to economy, the operating pressure being 200 pounds and the superheat averaging about 125 degrees at the throttles. The overall fuel cost per million gallons has been less than originally estimated.

The cost of the complete Parkway project was approximately $2,800,000.

When the station was first put in service, the promptness with which the red water troubles vanished was surprising. It had been apprehended that considerable time would be required for deposits in the mains to be flushed out. Actually it was only a day or two before the water at all taps was clear.

SHEAHAN STATION

In fundamentals, although not in detail, the Sheahan Station is a counterpart of the Parkway Station. It is being built for an ultimate capacity of 30 m.g.d., except that wells and mechanical equipment for only 15 m.g.d. are now being installed. The buildings and the initial group of wells all will be on a single L-shaped tract of land, the buildings being at the angle of the L and the wells extending out each arm for roughly a half-mile. Provision is made in the design

for future extension of the well system to greater distances in several directions.

All of the high service pumps will be steam turbine centrifugal units, instead of part being cross-compound engines as they were at the Parkway Station. The past 9 years advance in efficiency of the turbine type coupled with their other advantages, made them preferable under local conditions. The compressors will be steam-driven reciprocating units as before, but the units will be 50 percent larger and one less in number, and will be capable of somewhat higher air pressure. The secondary pumps, lifting water to the aerator, will be electrically driven by power generated in the station. The Parkway method of driving the secondary pumps by water turbines was less suitable here because the available driving water head is lower and the secondary pumping head greater than at the Parkway Station. The speed and delivery of the secondary pumps will be varied by varying the frequency of the generators, between the limits of 54 and 63 cycles. This plan is unusual but not original.

The design of the boiler room equipment has been improved, as compared with the Parkway Station, to prevent the escape of fine dust from the coal, and to eliminate oil in the condensate.

A pleasing feature of the Sheahan Station layout will be the symmetrical design and arrangement of buildings. The approach driveway lines up exactly with the central or main entrance to the pumping station, which is the middle building. On the left is the filter plant and on the right is the aerator, with entrances facing each other and with the main faces having the same dimensions and design. Each of these two buildings rests, practically speaking, upon the corner of a clear water basin, the earth cover of which merges in with the grade about the building.

The filter plant of the Sheahan Station will be equipped to operate practically automatically, except for occasional inspection during each shift and necessary filter washing. There is no chemical treatment or chlorination and the filter rate will be controlled by a master controller in such way as to maintain the water level on the filters within fixed limits, regardless of variations in the total rate of flow through the plant.

The cost of the complete Sheahan project will approximate \$1,750,000. It is estimated that the same work would have cost nearly \$2,500,000 at the 1922 prices of the Parkway Station, and over \$3,000,000 at the 1928 price level.

WATER QUALITY AND TREATMENT

In the quality of its water supply Memphis possesses advantages enjoyed by few large cities. The water delivered to the consumers is uniformly clear, palatable, soft for both domestic and industrial uses, free from bacterial pollution, color, taste, and odor and of uniform temperature of about 68°F. throughout the year. These properties of the supply are due to the nature of the underground water and to the extremely simple aeration-filtration treatment which employs no chemical whatever.

Prior to starting up the Parkway Station, the principal supply, which was drawn from wells of the old Auction Avenue field, was unsuitable for general use on account of the iron content which averaged 1.9 p.p.m. Some individual wells of the old supply contained as high as 7.0 p.p.m., others were nearly free of iron. In addition to the objectionable iron content the underground water is naturally very high in carbon dioxide and this resulted in picking up from the mains additional amounts of iron to the great annoyance of water consumers, on account of the staining of plumbing fixtures and clothing washed in the water.

The water from the wells of the Parkway system, used since 1924, has a lower iron content than from the old system, yet enough to be distinctly troublesome were there no treatment for its removal. On account of the high carbon dioxide the water delivered to consumers probably would be as unsatisfactory as from the old wells were it not for the removal effected by air-lift pumping plus secondary aeration.

A distinguishing characteristic of the underground water of the Memphis supply is the total absence of calcium and magnesium sulfates and chlorides which go to make up the so called "permanent hardness" of most water supplies. The Memphis water on the other hand contains significant amounts of sodium carbonate the amount depending upon the stratum from which the water is drawn. Besides the two water bearing strata before mentioned there are others that have been tapped by wells in the Memphis region, although they are of less importance. Samples taken from successively lower water bearing strata show that as the depth increases the water becomes higher in temperature, iron, alkali carbonates and total solids, and lower in carbon dioxide, hydrogen sulfide and hardness. The implication is very plain that the water in passing underground is

subjected to the action of natural zeolites which replace progressively the calcium and magnesium salts with corresponding sodium salts.

The proportion of water taken from either stratum varies during the year, but roughly 75 percent on the average comes from the upper stratum. The comparative quality of water from the two strata as sampled at the wells after air-lift is shown by the following analyses:

	PARTS PER MILLION	
	Upper stratum 500 feet	Lower stratum 1400 feet
Temperature, °F	63	21.6
pH	6.4	7.2
Free CO_2	33.6	6.4
H_2S	Trace	0
Dissolved Fe	0.57	1.40
Alkalinity	60.6	85.2
Hardness	42.0	11.8
Total Solids	81.3	112.3

The carbon dioxide content of the upper stratum water is assumed to be about 120 p.p.m. based on tests of wells in various parts of the City equipped with deep well pumps. Objectionable amounts of hydrogen sulfide occur in some of the wells but not all.

Before the Parkway station was built experiments were carried out (see Eng. News-Record, 1923, Vol. 90, p. 874) to determine the type of treatment required. It was found that the water after aeration gave no deposit of iron within 24 to 30 hours and therefore sedimentation would be of no value for iron removal. It was found that filtration through sand was required dependably to eliminate the iron from the supply. The use of air lift pumping removes about 75 percent of the carbon dioxide contained in the underground water and oxidizes the iron so that it will be readily removed by passage through sand beds. However the carbon dioxide remaining after air lift pumping, amounting to 20 to 30 p.p.m., is still objectionably high and further reduction by secondary aeration is necessary. Accordingly both stations incorporate the two features of aeration and rapid sand filtration. The particular design of aerator, or strictly speaking "de-carbonator," used was settled upon on account of the relatively high loading per square foot of surface occupied, the ease of enclosing within a suitable structure and the flat efficiency

curve for wide variations of flow. In the Parkway aerators, the water is distributed evenly by means of flumes with perforated plate bottoms to 40 aerator stacks, each consisting of four superimposed trays holding crushed coke of suitable size. The overall head utilized for aeration is about 9 feet. By passage through the successive coke trays the carbon dioxide is reduced to 5 p.p.m. or less. The Sheahan station aerator will be fundamentally the same, although different in arrangement and detail of trays.

The filters at both stations are standard design rapid-sand filters. The rate at which the filters are run and the variations in iron content of the water handled seem to have no bearing on their efficiency for removing iron. Compared with the practice of filtering surface water the filter runs between washings are very long, averaging some 70 to 80 hours, with some runs in excess of 100 service hours. Of the 0.7 p.p.m. iron in the raw water as received at the Parkway Station a small portion is deposited on the coke of the aerators, but the filters catch the balance so that the water delivered from the station has not over 0.03 p.p.m.

The chlorinator installed at the Parkway station is for emergency safeguard only and has seldom been used. The installation includes also two dry feed machines for adding lime to the filter effluent with the idea that it might be advisable to eliminate carbon dioxide entirely, but such treatment has been found unnecessary and the machines are never run.

The Parkway Station was designed and construction supervised by Fuller and McClintock, Engineers, of New York City. The engineers for the Sheahan Station were Fuller and McClintock and T. H. Allen, Consulting Engineer of Memphis. The architects for both plants were Jones, Furbringer and Jones, of Memphis. In both projects the wells were drilled and equipped by the Water Department with its own forces, who also laid the pipe lines connecting the wells with the plant and the plant with the distribution system. This work was done under the general charge of Jas. Sheahan, General Superintendent for the Board of Water Commissioners, for whom the new station is named. Over 30 general contractors participated in the construction of the two stations.

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(9) CUNNINGHAM: (New water works), Public Works, (1923), 54: 371; Public Works, (1924), 55: 13; also in Fire and Water, Mch. 26, 1924, p. 591.
(10) CUNNINGHAM: New well supply at Memphis, An. Rep. Indiana Sanitary and Water Supply Ass'n., (1924); also Eng. Contracting, (1924), 61: 773.
(11) DONALDSON: Early results from Memphis Water Purification Plant, Eng. News.-Rec., (1924), 93: 992.
(12) DONALDSON: Salient Features of the Memphis Water Supply, S. E. Water and Light Ass'n., May 21, 1925.
(13) MANTEL: Operating Results of Iron Removal Plant at Memphis, Tenn., Eng. News-Rec., (1927), 98: 855.

DISCUSSION

MALCOLM PIRNIE:[2] After seven years operation of the water supply and purification works designed by Fuller and McClintock the City of Memphis is now completing additional purification capacity identical in design with the original works. This is an excellent testimonial of the value of careful preliminary studies such as were made ten years ago under the direction of Wellington Donaldson to determine the best method of purifying the water as collected from wells by means of an air lift system.

At the time Mr. Donaldson was making studies to find a method for the removal of iron he investigated every possible method that might be applied economically to this water. I am convinced that he selected from the methods he investigated the best means of removing iron from this particular supply after it had been determined to collect the water from the wells by means of the air lift.

Another method can be applied very economically to small supplies where the content of iron in the water is materially higher than it is in the Memphis supply. It has been used successfully in several plants and requires a relatively small area for treatment works. Controls can be easily hooked up with the pumping equipment so as

[2] Consulting Engineer, New York, N. Y.

to operate automatically with the pumps delivering from the wells and the pumps delivering the iron-free water to the distribution system.

This principle was used at Long Beach, Long Island, in 1922, again at Stuart, Florida, and later at another plant installed at Punta Gorda, Florida. The iron in the raw waters delivered to these plants ranged from 7 to 19.6 p.p.m.

The principle of the treatment involved the use of surface forces in a deep bed of graded gravel to decompose the ferrous salts. The flow of water is upward through the gravel after regulated aeration to admit sufficient oxygen to allow the iron to change from the soluble ferrous state into the ferric state. The carbon dioxide weakly bound to the iron in ferrous bi-carbonate form is repelled by the wetted gravel surfaces and the iron is attracted and deposited on these surfaces. The water is introduced at the under-drains and allowed to flow upward so that gravity aids in holding back the deposits from the water passing through.

The beds are washed by simply opening and draining quickly at the bottom, allowing gravity to remove a large proportion of the accumulation. The large voids in the gravel beds serve both in the capacity of a settling basin and a removal filter, thereby conserving space to a minimum.

One thought in connection with highly charged iron waters is that it is advisable to keep air away from the waters in ground water bearing strata supplying the wells. If air is allowed to enter that water and gradually find its way back into the sand surrounding the collecting strainers, the iron will follow the same action that it does in the contact bed and be removed, filling the voids of the sand surrounding the collecting screen.

At Stuart, Florida, with direct suction on the wells, a plan of keeping the well pumps flooded by a vacuum system was installed in order to prevent any water that had received air from getting back into the sand surrounding the well strainers. In that way the efficiency of the wells has been maintained very satisfactorily for such high iron content water.

DEVELOPMENTS IN METERING AND CONTROLLING EQUIPMENT

By Charles G. Richardson[1]

Recent progress in water works design has been accompanied by numerous and advanced requirements for automatic control to perform various functions previously assigned to operators. Mechanical supervision of the movement of important units has superseded the less dependable and less exact human oversight. Three examples have been selected as typifying the nature, variety and solution of such problems.

SEWAGE FLOW REGULATION AT NEW HAVEN, CONN.

Previous to the completion of the sewage settling and chlorinating plant of 40 million gallons daily capacity recently placed in operation at New Haven, Conn., a 60-inch sewer discharged raw sewage directly into the harbor. The problem set up by the engineers was to deflect automatically the sewage through the new plant up to its capacity, but to guard against overloading the plant during heavy storms. Figure 1 illustrates diagrammatically how this was accomplished. Gate house A contains a large hydraulically-operated sluice gate B, which ordinarily is closed, permitting all of the sewage from the city to pass through the 48-inch pipe to the plant. On the outlet side of the plant at D is a 48-inch Controller Tube with hydraulically-operated single vane valve E. The effluent from the various clarifying tanks overflows into channel F, then passes through the Controller Tube and directly into the harbor. From the hydraulic cylinders of both gate B and valve E small lines, carrying about 35 pounds water pressure, lead to pilot valves G and H inside a special Venturi Register in a register house above ground, the mechanism of which is actuated by floats in two open vertical float pipes connected respectively to the inlet and throat of the Controller Tube. The distance between valves B and E is about 400 feet. The pilot valve pistons are actuated through proper gearing and cams by

[1] Builders Iron Foundry, Providence, R. I.

the rate indicator dial shaft of the Register. Up to a rate of 30 million gallons per day the 60-inch sluice gate B is closed and all of the sewage flows direct to the plant. When this rate is approached, however, an electric contact in the Register flashes a warning red light in the superintendent's office and pilot valve G moves to open main gate B, allowing part of the flow to pass directly into the harbor through the 60-inch main. Should the rate through the 48-inch line continue to increase, approaching 40 million, pilot valve H moves to throttle the Controller Valve E, automatically preventing a greater rate through the plant. When the storm subsides the original positions of the two main valves are restored. Simple adjustments are provided at the Register for setting the limiting flow rate values through either or both mains as local conditions may dictate. Besides indicating the rate of flow and giving

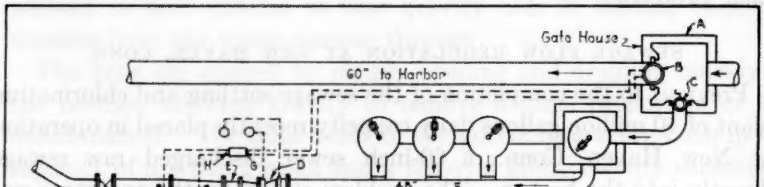

FIG. 1 VENTURI SEWAGE RATE CONTROLLER

the total quantity, the Venturi Register furnishes chart records of the flow rate, also of the time and sequence of events.

WATER LEVEL CONTROL AT ALBANY, N. Y.

At Albany, N. Y. a control problem proved rather formidable for a time. It was desired to maintain the water level on the filters within ±2 inches, and yet not permit such a wide fluctuation in raw water supply that it would be impossible to properly proportion the introduction of chemicals.

Raw water comes to the plant through a 60-inch main containing a Venturi Tube A with 30-inch throat and in the outlet cone a 36-inch Dow hydraulically-operated valve (fig. 2). Two hundred and fifty feet distant the water reaches a bank of aerators, then from the aerator basin flows by gravity through one to three mixing basins to the filters. The control equipment consists essentially of a large

float B in chamber C connected to the influent channel supplying the filters. A cable from this float leads to a pulley actuating one side D of a differential mechanism, mounted in metal case E, and operating pilot valve F, which is in turn pressure-connected to hydraulic cylinder G of the Dow valve (fig. 2A). The other side of the differential mechanism is connected to the stem of hydraulic valve G.

To illustrate the action, imagine the output of the filters is increased. The water level on filters begins to descend, changing the elevation of float B, whose movement, through the differential mechanism, causes pilot valve F to admit water pressure to hydraulic

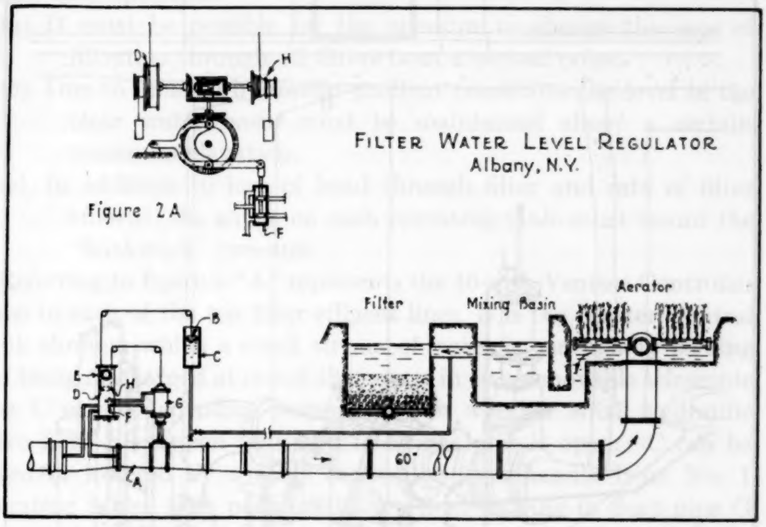

FIG. 2

cylinder G, opening the Dow valve further and thus increasing the raw water rate. The valve, however, moves but a small amount before its motion is arrested by returning the pilot valve to its neutral position. It is apparent that to obtain proper level control the gearing of the differential mechanism has to be carefully calculated for proper relation between float and control valve travel, since both actuate the pilot valve. At Albany the long distance which the water had to travel from control valve through aerator basin, mixing and settling basins to filters resulted in a hunting action, since the increased supply through the valve did not immediately reach the filters. This resulted in a fluctuating flow from 0 to

20 or 25 million gallons per day in cycles of one hour, entirely unsuitable for chemical feed control. Manual throttling at the aerator nozzles proved partially effective, but impractical for a changing filtered water demand. The solution was discovered in regearing the differential mechanism to give a much quicker response of the pilot valve to small increments of control valve movement and this was

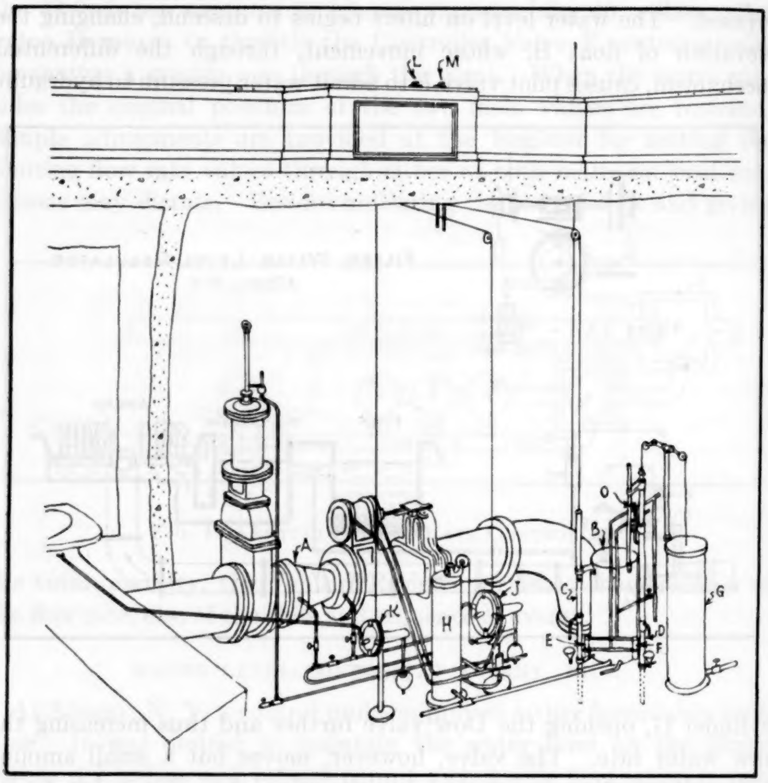

FIG. 3. VENTURI EFFLUENT CONTROLLER WITH MASTER CONTROL

also made adjustable through the use of a tapered grooved pulley H (fig. 2A) receiving the cable from control valve stem. After the changes were made with one mixing basin in service and an effluent of 12 m.g.d., the raw water rate varied only from 11.2 to 13 m.g.d. and with an effluent rate of 22 the variation was 22 to 23.5 m.g.d. With all three mixing basins in service operating in series, which is

the normal arrangement, and with an effluent rate of 22 m.g.d., the variation in raw water was from 20 to 22 m.g.d. in regular cycles of about ninety minutes. At the same time the water level on filters was maintained within 1.08 inches.

MASTER EFFLUENT CONTROL AT OTTAWA, ONTARIO

Perhaps the most interesting problems of automatic flow control have been associated with the marked advance in the design of rapid sand filtration plants serving large populations. The new Ottawa, Ontario, Plant is a conspicuous example. Three requirements were specified:

(*a*) It must be possible for the operator to change the rate of filtration through all filters from a central point.

(*b*) Due to unusual hydraulic gradient conditions the level in the clear water basin must be maintained above a certain minimum elevation.

(*c*) In addition to loss of head through filter and rate of filter effluent, the gauge on each operating table must record the "backwash" pressure.

Referring to figure 3 "A" represents the 16-inch Venturi Controller Tube in each of the ten filter effluent lines, B is the Master Control Tank through which a small stream of water is continually passing and being discharged at one of the spouts in either movable telescopic pipe C or D, depending respectively on whether small hydraulic valve E or F between tank and telescopic pipe is open. C can be raised or lowered by a cable connection to a handle L on No. 1 operating table; D is positioned by a float moving in float pipe G connected to the clear well. Master tank B is connected to float pipe H and nine other similar float pipes, one at each Controller. The floats in these pipes, through a cable connection to Diaphragm Pilot Valve Unit J, position the pilot valve can slide on this unit; the latter actuates the rate of flow pen of the Gauge on operating table. Pendulum Unit K actuates the loss of head and backwash pen. If the operator desires to increase the filtering rate, he moves handle L, which raises telescopic pipe C. The level in the Master Tank consequently rises, also, correspondingly, the level in the individual tanks, such as H, causing a movement of the pilot valve on Unit J to admit water to the opening side of the controller valve cylinder until the desired rate is obtained and shown by the Gauge, when the resulting pendulum movement of Unit J restores the pilot

valve to its neutral position and stops further movement of the Controller Valve. If the operator observes, from a large dial, centrally located, that the clear well level is approaching the minimum limit allowable, he turns 4-way cock handle M, which is pressure-connected to the small valves E and F, to switch the flow to spout on telescopic pipe D. It is obvious that the clear well level then assumes the master control to increase the flow through each effluent line. As the float in pipe G rises with clear water level, the master rate is decreased and, when the safe level is obtained, the operator can again switch back to the previous master rate by simply turning handle M. (A more recent attachment O, indicated on figure 3, will make the change-over from manual master to clear well master and vice versa entirely automatic, without any attention whatever by the operator.) Pendulum Unit K has only one pressure connection direct to the effluent pipe and close to the filter, as indicated. During normal filtering, assuming constant level on filter, this pressure is exactly proportional to the increasing loss of head and is so transmitted through the Pendulum Unit to the Gauge. When the main effluent valve is closed for washing, this pressure becomes the backwash pressure, which, at Ottawa, will be proportional to the sand expansion during the washing period, and hence significant for controlling velocity of wash.

The Gauge on each operating table at Ottawa is elaborate and unique. It is entirely of bronze, satin finish, and of architectural design in harmony with the tables and the operating room in general. The overhanging hood conceals electric lamps for illumination. The chart is for a month's run, with 24 hours exposed, although portions can be cut off at any time. There are two pens: the upper to show the rate of filtering, the lower the loss of head and backwash pressure.

DISCUSSION

M. M. Borden:[2] Mr. Richardson's excellent paper draws attention to the increasing number of special problems involving the measurement and control of sewage and water in modern works.

In some cases, satisfactory solutions are obtained by the intelligent application of standard equipment, but in most instances special constructions are necessary.

[2] Simplex Valve and Meter Company, Philadelphia, Pa.

The state of the art of water control and measurement is such as always to insure excellent results, as illustrated by Mr. Richardson, when the problems are treated by sound principles and in the light of broad experience.

One of the difficulties attending such special work, however, is the lack of volume demand. This results in a belief on the part of the buyer that the selling price is too high, despite the more certain knowledge on the part of the manufacturer that the cost is too large a portion of the price which is charged.

CHANGES IN COST OF WATER WORKS LABOR AND MATERIALS

By William W. Brush[1]

In 1918 a committee of the American Water Works Association on "War Burdens of Water Works in the United States" made an exhaustive report setting forth the changes in cost of labor and materials based upon data received from about fifty municipally and corporately owned water works plants in the United States. These data was grouped on a geographical basis under the captions "Western," "Central," "Eastern" and "Southern." In this report it is stated "It appears clear that average pre-war prices will never again be realized and that the purchasing power of money has declined permanently the world over, as a result of the war, and will never be fully recovered. Present prices, which on most water works materials are double pre-war prices, and on labor 25 to 50 percent greater, will probably not hold permanently after the war. Nevertheless, the old prices will not return as a whole."

The report is signed by George W. Fuller, George A. Johnson and Leonard Metcalf, Chairman. Mr. Metcalf continued to gather and present these data for 1919, 1920 and 1921, but due to his death this work was discontinued until it was in part recently undertaken by Mr. Malcolm Pirnie on a different basis than that followed by Mr. Metcalf. Mr. Pirnie has kindly consented to allow the writer to present the results of some of his investigations expressed diagramatically and those curves show that the changes in the past two to three years have been nearly as drastic in the decline as they were in the rise during and following the war period (see figures 1 and 2). Many of our association members are vitally interested in the changes that take place in the cost of labor, materials and other items that form what are termed construction costs. There is one item of cost which affects all of us and to which we are inclined to give a position of less importance than it deserves. The wide fluctuations in the cost of borrowed money is shown in figure 3 and if this diagram could

[1] Chief Engineer, Water Supply, Department of Water Supply, Gas and Electricity, New York, N. Y.

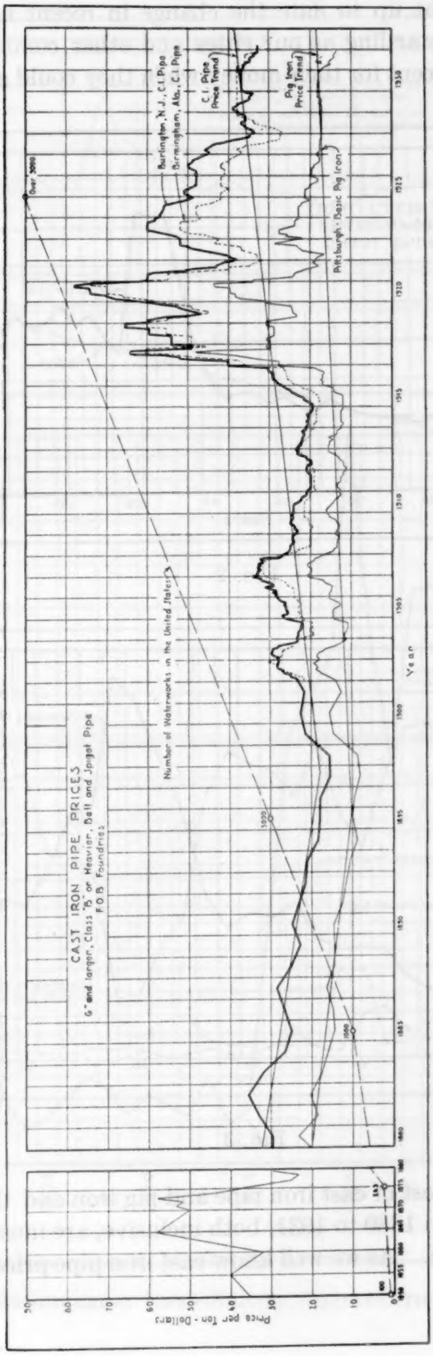
CAST IRON PIPE PRICES
6" and larger, Class 15 or Heavier, Bell and Spigot Pipe
F.O.B. Foundries
Number of Waterworks in the United States
Burlington, N.J., C.I. Pipe
Birmingham, Ala., C.I. Pipe
C.I. Pipe Price Trend
Pig Iron Price Trend
Pittsburgh - Basic Pig Iron
Year
Price per Ton - Dollars

FIG. 1

have been brought up to date the change in recent months would have been most startling as our cities and other communities have been paying 6 percent for their money when they could get it.

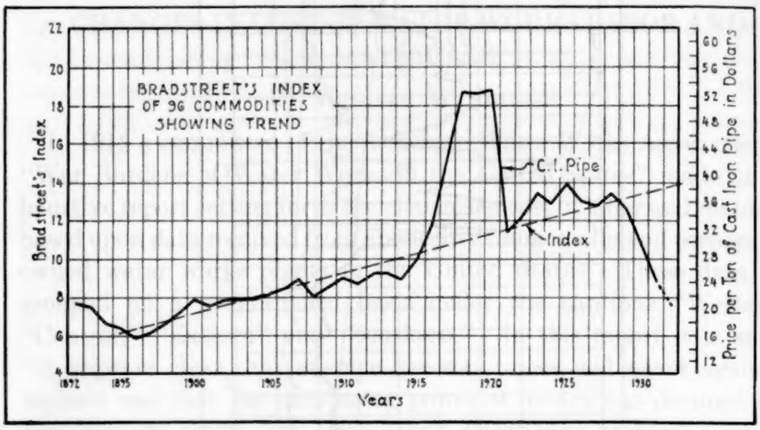

FIG. 2

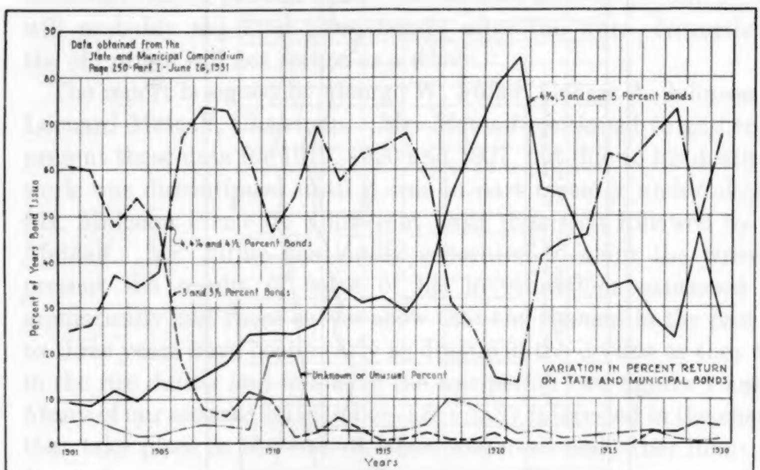

FIG. 3

The changes in cost of cast iron pipe and pig iron and their relation to one another from 1880 to 1931, both inclusive, are most interesting as shown in figure 4. As we well know cast iron pipe prices have been

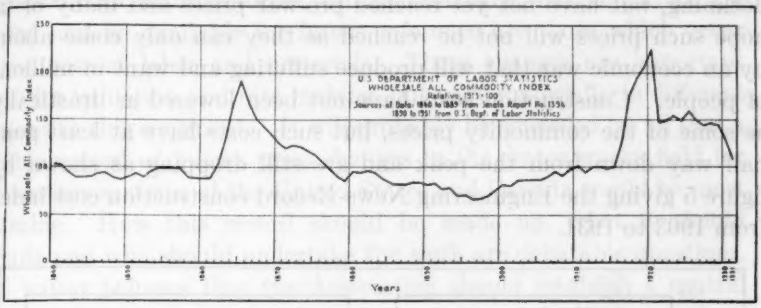

FIG. 4

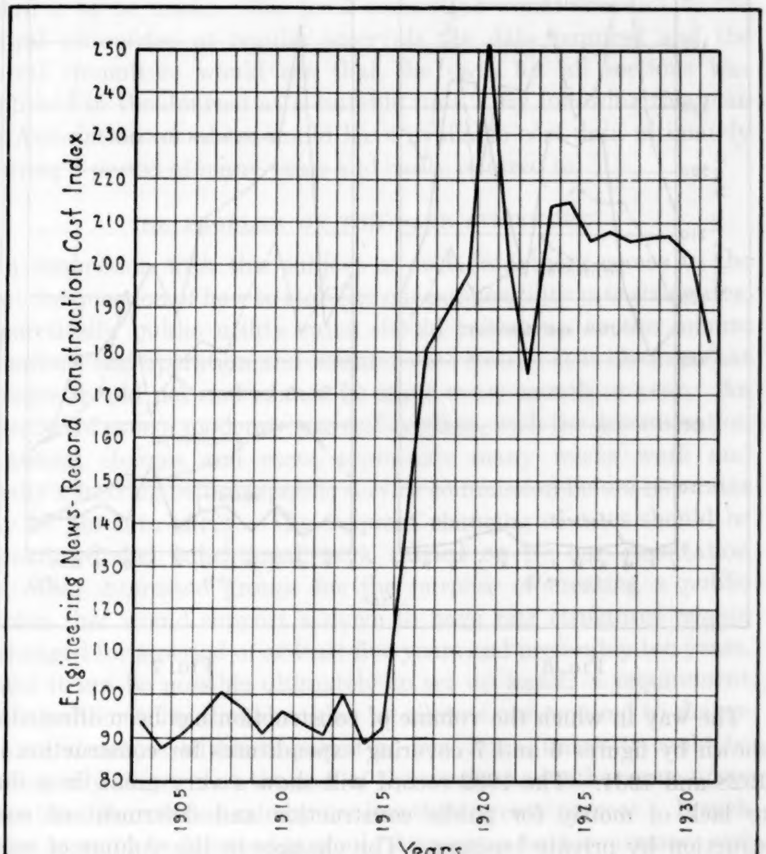

FIG. 5. ENGINEERING NEWS-RECORD CONSTRUCTION COST INDEX

declining, but have not yet reached pre-war prices and many of us hope such prices will not be reached as they can only come about by an economic war that will produce suffering and want in millions of people. Construction costs have not been lowered as drastically as some of the commodity prices, but such costs have at least gone half way down from the peak and are still dropping as shown by figure 5 giving the Engineering News-Record construction cost index from 1903 to 1931.

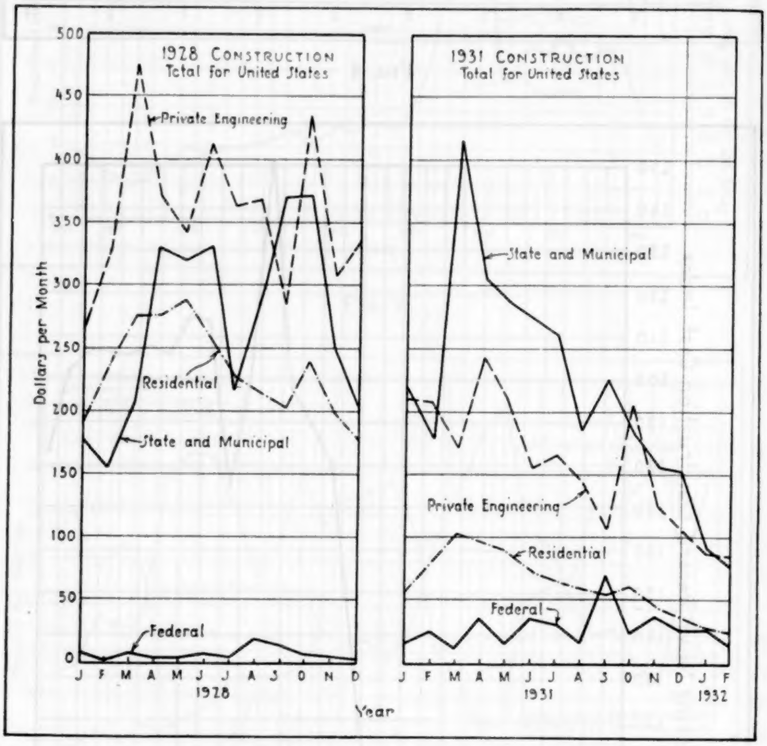

FIG. 6 FIG. 7

The way in which the volume of construction has been affected is shown by figures 6 and 7 covering expenditures for construction in 1928 and 1931. The 1932 record will show a very great drop due to lack of money for public construction and deferment of construction by private business. The changes in the volume of construction are of very immediate interest to engineers and water works

operators and manufacturers, but the changes that should be recorded and be available for future reference are those which occur in the cost of labor and materials. These changes are important bases for valuation and rate making, but vary materially in different sections in this continent. It will probably be conceded by all that a reliable record of yearly price changes in the water works field for the various sections of the United States and Canada would be most desirable. How this record should be made up, what it should include and who should undertake the work are debatable questions. The writer believes that the Association should establish a central steering committee to have charge of securing and tabulating these price changes with local committees in each section for which a record is to be made. The local committee would transmit to the central committee at regular intervals the data required and the central committee would see that the data for all sections was published in the Journal at a suitable time. By following this plan the Association members would have available cost data ultimately covering a period of many years and easily referred to.

THE PROBLEM OF THE RATE STRUCTURE

In connection with this subject of cost data there comes to the front the question of how to stabilize our public utility rate structures. Theoretically public utility rates should reflect up to the minute valuations and operation and maintenance costs. It is obvious that changes should not and cannot be made every month or year. An appraisal of even a moderate size utility plant, with the determination of annual charges and costs, represents many weeks work and usually a hearing before a public service commission before new rates may be put into effect. The frequent changing of rates should be discouraged and educational work carried on by our Association and other interested groups for the purpose of creating a public opinion that would support a move to have rate structures remain unchanged for a period of at least five years and preferably ten years. Might it not be possible ultimately to set up legally a requirement that a public utility should neither be allowed nor required to change its rate structure within a period of five years, unless it could be conclusively shown that the cost of the service had either gone up or down from the existing rate by not less than twenty percent. If such a plan were to be followed it would discourage both consumers and public utility officials from attempting to establish new rates on an unreasonably slight change in total costs.

THE NECESSITY OF WATER WORKS BETTERMENT

There is a subject which properly comes under the title of this paper that deserves the support of all of us and yet one which it is most difficult to promote under present financial stringency. The writer refers to the undertaking of water works betterments to improve the service rendered to the consumers and to aid in the economic recovery. All of us who have charge of water works plants can enumerate various items of capital expenditure that have not been done, but which should be done in the interest of service to the community. At present contract prices for construction work each dollar will buy nearly twice as much as it would have bought three to four years ago. It is certainly good business to spend the dollars now if the dollars can be obtained. Unfortunately, the bankers of this country have not been able to view this part of the water works superintendent's plan favorably and unless the money can be borrowed at a reasonable rate of interest no betterment program will have the support of our financial institutions.

The following is a list of improvements that should be undertaken on a large scale in most of our water supply plants.

1. New and larger lateral mains to replace mains of inadequate size due to small diameter or tuberculation.
2. Additional trunk mains so that any trunk main could be out of service for repairs without seriously lowering the service furnished for domestic consumption and fire protection.
3. Addition of valves and hydrants including valves on every hydrant branch so that the service would be on a high standard of efficiency.
4. Development of additional source of supply so that there would be sufficient water to meet all legitimate demands even if the most severe drought which comes once in one hundred years were to now visit us.
5. Softening hard supplies which are of satisfactory sanitary quality but are not satisfactory for either domestic or manufacturing use due to a high mineral content.
6. Improvement of the taste of the water by treatment or by constructing filters or other appurtenances that will remove a disagreeable taste that may be present at times in the water and interferes with the consumer's enjoyment and use of the water.

In the plant under the writer's charge many millions of dollars could be advantageously spent in improvements and extensions that come under the headings just given. It is most unfortunate that instead of now endeavoring to speed up such work and employ thousands of workers the city authorities have felt compelled by the pressure from the bankers actually to reduce the scale of capital expenditures from about $6,000,000 to $1,000,000 yearly.

In closing the writer submits to the Association a motion that a committee composed of the President and the Chairmen of the Water Works Practice and Publication Committees appoint a committee to collect and record changes in the cost of water works labor and materials in the various sections of the United States and Canada.

DISCUSSION

W. A. HARDENBERGH:[2] Mr. Brush has discussed this paper so well, in his usual fashion, that I am not sure that I can add very much to it.

"Public Works Magazine," being desirous of finding out from the superintendents, themselves, just how much the cost of construction has gone down, sent a questionnaire to many of them.

We asked information on the cost of water main construction for 1925, 1929, 1931 and 1932. We have received so far replies from about five hundred cities. Of course, not all of these cities have laid pipe in all of these years. Some of them have cut down very drastically on the amount of construction during the past two or three years.

A number of them did lay pipe in each one of those years and had accurate cost data thereon. However, in checking over them it was found out that it did not make much difference in the final results whether we took only those cities that showed the records for these whole four years, or whether we lumped them altogether. The question asked was: "What was the average cost per foot, including pipe, excavating and back filling, but excluding hydrants, valves, etc., for 8-inch pipe?"

Replies from about one hundred and fifty cities from all parts of the country showed, in 1925, an average price of $2.21; in 1929, a price of $2.02; in 1931 the average price was $1.75; and in 1932, $1.42.

[2] Vice-President, "Public Works," New York, N. Y.

These are not separated by sections. Of course, the reports from the cities in the South, where we have less cover, and perhaps less labor costs, will be lower. But we were not trying to get anything except the actual relation between the years mentioned. Assuming 1925 as 100%, 1932 costs are now 64.3 per cent, or taking the average of three other years, 1925, 1929 and 1931, the cost for 1932 is 71%. This agrees quite closely with 1932 construction costs, as shown by Mr. Brush. The "Engineering News Record" sets costs now at about 76 percent of the average of 1923 to 1931.

We got a number of reports of other sizes of pipes, on which the percentages ran almost the same. As has been pointed out by Mr. Brush, and by others, these reduced costs make construction economical now, even though money costs more to get. Harrison P. Eddy has shown that a project which three years ago would have cost a hundred thousand dollars can now be built for about seventy-six thousand dollars if, as Mr. Brush says, you can get the money; and even if a rate of 5.5 percent must now be paid as compared to 4.5 percent at that time. The annual interest charge will be slightly less, and of course, the sinking fund will be something like 23 or 24 percent less.

It seems also that, at this time, when we are having rather unusual developments in water treatment, there are great possibilities in better engineering by employing modern equipment and ideas, so that the initial investment may be less, or the operation costs smaller. By this term "better engineering" I do not mean to say that there is not now adequate and excellent engineering talent available. What I do mean to say, and this is based on considerable observation of my own, is that there are too many communities willing to secure second rate engineering service, because they can get it a little bit cheaper. Almost invariably in the end they pay several times over for the small saving that they may make by getting a cheaper engineer.

It is easy to go into a community that needs an extension of its service and facilities and to sweep all the old things into the discard; or to design a stock job based on a somewhat similar one of last year, or the year before that. It will fit, in a way, the conditions that are found. But it takes experience, and skill, and hard work, and lots of thought and management and planning to utilize the salvageable portions of the old plant; or to design a new plant that will take care of legitimate loopholes for cost reduction and yet meet fully the needs of the community, not only for now, but for the future.

As one instance of good engineering, I can mention a recent Middle West installation where, by the employment of modern mixing equipment in connection with coagulation, it was possible to avoid the construction of a new coagulation basin. The old basin had been laid out in such a way that it could not be enlarged. The new equipment saved about one-third of the alum that they had previously employed.

Mr. Brush spoke about the difficulties of financing in these days. It probably is the major problem today in water works construction. In some cases we may pay for construction out of income. Several sewage treatment plants, where normally there is no income, have been built recently, out of income, notably in Ohio. There is no reason why water works construction cannot be carried on in the same way.

To do this requires a master plan so that each portion of the plant as it is constructed should fit in with the next part. That is what is meant by stage construction. Put in one necessary unit now; another one next year; another, year after next; or whenever money becomes available. The highest type of engineering is required or the entire purposes of the plan may be defeated.

There is a great need for the carrying out of water works construction at this time in order to meet the deficiencies caused by the postponements of construction during the past two or three years and to meet the needs that have been shown up by the dry weather conditions that we have passed through in the last two or three years.

The stage construction plan might be especially adopted to this picture and to this particular period, in many cases. It is not necessary to sell bond issues. Quite frequently the money can be raised locally. A small community near my home town is now doing that. They are laying a pipe line they have needed for some years. They have made arrangements to handle all the finances through the local banks. That obviates the necessity of selling a bond issue and removes that difficulty from the whole proposition.

It is probable that, where a careful financial control has been accomplished in the past so that the bankers can be convinced that the investment will be safe and they will have some way of getting their money out of it, this can be accomplished.

There are occasional opportunities for economy in design also. Perhaps temporarily we can omit certain details designed to make our operation easier. This is especially true of some of the smaller plants where not all of the time of the operator of the plant is spent

in routine duty. We may say, of course, that a small plant that can not afford able, or a well trained operating head, needs complete apparatus for automatic control. But often the sinking fund represented by the additional amount necessary will pay the salary difference necessary for a good operator, and the initial saving is pretty important. People do not like to vote for bond issues and the banks do not like to advance money unless they have to.

In referring to the sewage field again, I know of a sewage plant where for the removal of the dry sludge from the sludge beds, a complicated industrial system that cost something like five or six thousand dollars, or a little more was installed, although the volume of sludge to be removed each day was less than one yard of dry material. This could easily have been handled by the operator in his spare time.

THE INFLUENCE OF WATER WORKS ON FIRE PROTECTION CLASSIFICATION

By George W. Booth[1]

As communities develop, the destruction of property by fire is no longer an individual affair; it becomes a neighborhood concern, as the burning of one structure may, and often does, mean the exposure of other buildings and ultimately a general sweeping conflagration.

It is to overcome and prevent these conflagrations, involving a number of buildings, several city blocks or an entire community, that fire fighting facilities must be provided, and maintained in ever present readiness. But these fire fighting facilities, which include water supply, fire departments, fire alarm systems and police forces, are not the only factors influencing or retarding serious fires or conflagrations. Early in the history of nations it was understood that wide open spaces, blank brick or stone walls, use of incombustible materials for building and a limitation of area and height of structures were of great value in restricting the spread of fires. In more recent years the probability element has assumed a growing importance and correction of conditions which makes a commodity or device a fire hazard has become one of the features which must be considered in evaluating fire protection. Thus laws and ordinances dealing with explosives, combustibles, oils, gases and hazardous processes are necessary.

Water has always been recognized as the principal agent available for extinguishing fires. It has great cooling value, absorbing a large number of heat units when it vaporizes into steam and thus reducing the burning material to a temperature below its ignition point, as vaporization takes place at 212° and the ignition temperature of combustible material, oils, and gases is above 500°. Water and its vapor also exclude or dilute the oxygen content of air to a degree where combustion cannot continue.

Water is of the highest value as an extinguishing agent because of its universal availability, its ease of storage, the readiness with which it can be transported and handled, and last, but not least, its low cost.

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For incipient or small fires many other materials are used and their value is recognized by fire underwriters as first aid appliances. But as a principal means of combating serious fires and of stopping conflagrations there has been no substitute found for water.

This brief introduction has the object of indicating that, although water is an essential in fire protection, it is only one of several factors to be considered; it therefore follows, especially in the many cases where the water supply is already better proportionally than the other features, that material and costly additions to the water supply system may be made without accomplishing a general betterment in the insurance classification, and there may be no general reduction in rates.

It can be readily understood that where a fire department is seriously inadequate, as in hose, nozzles, pumping engines and men, the addition of more capacity to an already mainly adequate water system cannot materially improve general protection.

No scientific determination can be made of the relative value of the various features which go to make up this general scheme of fire protection, but after careful study, and an averaging of the opinion of many men who had considered the subject, it was determined that water supply should be assigned a value comprising about 34 percent of the total.

Insurance rates are determined on the basis of obtaining an average return in premiums, over a period of years, such that the fire insurance company can pay overhead expenses of the business, including commissions, taxes, etc., and also meet the losses or claims made upon the company; in addition, a profit is due the stock holders who are risking their money in the business. These losses vary from year to year, with excessive peaks in the loss curve every few years, and with marked variation in the various parts of the country.

Because of these variations in losses, and the fact that, by State legislation, nearly every State is an individual underwriting unit, there can be no definite fixed value set by the fire underwriters on any improvement in a waterworks plant. Even improvements which may result in a change of one class cannot be given a monetary value, nor so much as a fixed percentage of the premium income which will be correct for all states. This is in part due to the difference in rating schedules which have been adopted in the various states. These rating schedules materially differ in their fundamental makeup and from time to time have been changed, often by action of the State

supervising authority. It is thus no more possible to predict or assure a definite insurance reduction, in either cents or percent of premium, than it is to foretell the actual or relative reduction of the retail cost of a suit of clothes which would be brought about by the lowering of the cost of wool, or in the labor cost.

This analogy might be extended to a consideration of reduction where quality is concerned; a 10 percent change in the price of raw material may affect suits selling for $20, but have no influence in the sale price of $100 suits. Similarly, such plants as sprinklered risks and some extensive manufacturing plants cannot be influenced in their premium by a general reduction of rates due to improvements in public water supply.

It is usually possible, however, when definite plans for improvement are proposed, to determine with a reasonable degree of accuracy how many points of deficiency will be removed. It is also possible to give an approximate estimate of the effect a change of one class will have on the base rate of the city; this in general is about 5 percent. But to determine the actual saving to the citizens is quite another problem. As stated above, some classes of insured property will not be affected by a change in class; since their construction and private protection are such that public protection has little bearing on the final rate. The rates on dwellings, with fewer classes of protection, do not always change with a change in other classes of insured property.

Two major items must be taken into consideration in connection with this study of credit in insurance rates for improvements to water supply. The first is that, irrespective of all other conditions, the insurance companies must have an income sufficient to pay losses and the cost of doing business. Further that fire insurance is a business in which averages form the fundamental basis and although certain communities, or a state as a whole for certain years, may have a widely varying fire loss record, the average over a term of years must be the determining factor. From this consideration it is evident that individual communities may materially improve their protection, but if this is not followed, as is sometimes the case, by a lowered total fire loss for the state there may be a necessity for a general increase in insurance rates. In such instances the cities which have made improvements receive an indirect benefit, especially if there has been an increase in population which calls for correspondingly greater fire protection; under these conditions the community which has more

than kept up-to-date in its fire protection, although not receiving a reduction in rates, does not get an increase, and thus benefits. There may be, on the other hand, such a betterment of the state fire loss record as to result in a general reduction in rates throughout the state. The state of Ohio furnished some years ago an example of such a rate reduction, brought about by a combination of effective fire prevention activities and general improvements to fire protection facilities.

The second major item is that the heavy burden of cost of frequent rerating of a community is not justified. This cost is large and must be passed on to the citizens as part of the overhead of the business. Here again the question of averages is a factor, and therefore, it would be impractical for the insurance business to attempt to reclassify and rerate any given community at too frequent intervals.

Certain of the larger cities of the country have their general fire protection so far advanced that there is little to be done which can be given recognition by any general empirical formula, such as a grading schedule of fire protection. In such cities the volume of insurance is so enormous that it is the relation of losses to premiums, over a period of years, which must be the controlling factor, and not a classification giving a relative figure to other cities in the State or the country at large.

Even in the smaller cities there may come a time when the water supply becomes so reliable and adequate there will be no further credit which can be given to improvements in the water supply. Obviously it would be incorrect to offset a serious deficiency in the manpower of the fire department by a credit for more water than is believed usable at the most severe conflagration; nor can an excess of hydrants replace a deficiency in fire alarm boxes.

This condition of a good water supply in which little additional credit can be obtained for improvements is fairly common in New England and some of the other older portions of this country, and especially in the areas where topography is such that large storage can be obtained near the city.

This storage, as part of the impounded supply of the run off of the water shed, will seldom be depleted to an extent where it is less than the fire demand for 10 hours, which has been proven to be a safe estimate of the duration of a severe fire. Long before this degree of depletion becomes a factor from the standpoint of fire protection it is considered as a warning of probable shortage of domestic supply, and to look to the future extensive new work must be undertaken.

Little of this can be reasoned as due to the needs of fire protection and therefore no monetary return should be expected from the fire insurance interests.

Much extensive work is done to provide water of better quality; the expenditures are heavy, and, too often, in pushing bond issues it is made to appear that this will greatly improve fire protection. Such is more than likely not correct, as the installation of filters, the extension of supply tunnels or supply lines to a safer source, or the entire abandonment of a polluted source for one which can be protected do not always mean better protection; in fact several cases are on record where a filter plant has been installed of insufficient capacity to meet maximum consumption and fire flow, where previously pumps took suction direct from a river of unlimited capacity.

Many instances are also on record where to obtain a more suitable supply an entirely new location has been developed, often on the opposite side of the city, with the result that much of the large pipe is no longer of value and there must be laid many miles of new supply line to assure an equal fire supply. Revamping of a pumping station, the substitution of electricity for steam and numerous other changes costing large sums of money are often advantageous from the standpoint of economy of operation; but do these have much bearing on fire protection?

The answer in most cases is that fire protection is little concerned with economy of operation, since the cost of operation during the short-time fire demand is a factor which does not justify first consideration.

PORTION OF WATER SYSTEM CHARGEABLE TO FIRE PROTECTION

There can be no question but that hydrants are an item the cost of which can be directly assigned to fire protection. Likewise a portion of the cost of the distribution system, but as the community becomes larger the proportion which can be allocated to fire protection becomes less. Probably in even the largest of our cities some of the cost of minor distributors is chargeable to fire protection, but even this is becoming less as domestic consumption continues to advance and the demand for a more constant supply in the upper stories of buildings becomes more pressing. Few extensive residential areas, where much lawn sprinkling is done, can be supplied without periodical complaints from the owners if the distribution system is not well gridironed and in part of 8-inch pipe. The main arteries and second-

ary feeders of many of the larger cities are of the sizes determined mainly because of consumption demand. No part of their cost can be considered as due to fire protection where the excess flow required to meet peak hourly demand is greater than the probable fire flow for a serious fire. Numerous instances are on record where extensive areas have been seriously affected in pressure and volume available in the sprinkling hours of the summer, but fire flow tests have indicated ample fire protection during normal rates of domestic consumption, and even during rates equal to the maximum twenty-four hour consumption. As experience has shown that during an extensive fire consumption will almost never exceed the maximum twenty-four hour rate of demand it is obvious that where a condition occurs, such as has been described above, any material addition to the arterial system must be charged to the desire to render service free from complaints rather than to fire protection.

In connection with the subject of arterial systems it should be borne in mind that, with the exception of outlying factories, which often have extensive and generally adequate private protection, the section of a city or town requiring the greatest fire flow is in and around the business or downtown district. For this reason large mains looping around or extending through the outskirts of the city have comparatively little value from a fire protection standpoint, and in most instances a more satisfactory general strengthening of the distribution system can be obtained by running most of the large mains to or near the business district, and radiating from there to outlying sections.

There are certain features in the layout of a distribution system which are essential to reliability of service rather than to adequacy. These are the looping of large mains so that a break will not interrupt supply, and the use of sufficient gate valves so that too great an area will not be without water while repairs are being made following a break. These are important items from the standpoint of fire protection, but who would say they are not equally important to industries, hospitals, buildings using hydraulic elevators, and to the people as a whole who must have water to drink, cook with and bathe in? In like manner, fireproof pumping stations, reserve pumps, boilers and other equipment, to be used when the regular appliances are being repaired, are of great value to both fire protection and domestic consumption. It is not believed that the cost of any of these should be assigned to fire protection, except insofar as the fire demand exceeds the difference between peak hour and maximum twenty-four hour domestic rate.

TREND OF INSURANCE RATES

In fixing the insurance rate on individual buildings there are many factors other than those influenced by the effectiveness of municipal fire protection. Let us first analyze the trend of insurance rates generally during past years. In the ten years 1881 to 1890, the average premium per $100 of insurance written throughout the United States was $1.0347; the total yearly insurance covered only an average of $10,993,000,000.

Beginning with 1910, with an average premium of $1.0774 per $100 of insurance, there has been a steady decline in the rate per $100 until in 1930 it was only 0.7691. It is pertinent, in this connection, to note the total insurance covered in 1910 had increased to $43,124,000,000, which was 392 percent of the average in the period 1881 to 1890. The corresponding figure for 1930 was $151,349,000,000, approximately the same average rate of increase as in the previous 20 years.

What is responsible for this decrease in rates charged per $100 of insurance? There are several reasons which can be assigned: First, there is a lessened overhead cost resulting from a greater volume of business. Second, in 1910, of the total insurance written 60 percent was for one year and the remainder for terms of 2 to 5 years, while in 1930 only 47 percent was for one year; when it is understood that the term business is at a less total premium (the premium for a three-year policy is only 2½ times that for a one-year policy) it can be readily seen there would be reduction in the average rate per $100 of insurance written. Third, in the past twenty years most of the larger cities have adopted modern building codes, which require safer construction from the standpoint of fires; practically all new construction in the high value centers is fireproof, and while values have greatly increased the insurance rate has dropped to a fraction of that which applied to the building replaced. Fourth, in this period there has been a more general use of definite schedules for determining the rate on individual buildings, with the result that these properties receive credit for better construction and protection, and there is a general tendency towards decrease in the average rate.

The factors above mentioned will have, in many instances, a much greater influence on the individual building rate than will public fire protection facilities. There have been, moreover, material advances during the past 20 years in the various agencies of public fire defense. Fire Departments have shown much improvement in these twenty years; quicker and more effective work has been brought

about by automobile apparatus; there has been a general increase in manpower, and the science of fire fighting has greatly advanced.

Fire alarm systems have been improved and extended, and the telephone has become of great value in expediting the transmission of alarms of fire. All of these features must be given some credit in the general betterment of protection which might have a tendency to reduce the classification and thus the basis rate of a community.

THE EFFECT OF WATER SUPPLY ON INSURANCE RATES

It is not the intent to indicate by the foregoing that there can be little or no credit given in insurance rates for waterworks improvements. Such is not the case. With the introduction of a water system in a community, such as to provide a fire flow of approximately 500 gallons a minute, the community goes from the "unprotected" to the "protected" class, provided some form of fire department is organized and equipped. Such a change in rating greatly affects all property, including dwellings. Having reached the protected class, there are, in general, seven or more superior classes of protection to which the city may aspire. As respects dwelling rates, which are more basic in character, there are in most of the states only two to four dwelling classes.

In the Standard Grading Schedule of the National Board of Fire Underwriters there are 5,000 points of deficiency in the ten classes, which means that 500 points are necessary to change a class. Water supply constitutes only 1700 of these points so it is obvious that water supply alone can produce a change of about three classes; but as cities grow other protective features usually keep pace and credit given for water-works improvements aids in either maintaining a city in its class or in putting it in a better class.

As examples of betterment in insurance rate structure due to improvements in water supply, the following data are presented: For 58 cities graded by engineers of the National Board of Fire Underwriters during the past two years on which gradings had been made approximately 10 years before, 44 showed improvement in the water supply and 14 deterioration. Of those showing improvement, the average of the old gradings, as applying to the water supply only, was 501 points which corresponds to a deficiency of 29 percent and of the new gradings, 329 points, or 19 per cent, a gain of 172 points. with a maximum improvement of 710 points. Of those showing deterioration the old average was 435 points, or 26 per cent and the

new 541 points, or 32 percent, a loss of 106 points, with a maximum loss of 659 points. The average of all the 58 cities was 485 points ten years ago and now 380 points, an improvement of 105 points, which is 6 percent of the 1700 points of deficiency applicable to water supply.

Of about 400 cities graded by the National Board during the past 18 years 45 have a water supply less than 10 per cent deficient, seven of these having separate high pressure fire systems. Twenty have less than 100 points of deficiency, with a minimum of 27 points.

Some individual cases showing the kind and extent of improvements made are cited below:

A city in Ohio, with a population of about 43,000, when first graded showed 1044 points of deficiency in the water supply. Subsequent improvements in the system included an equalizing and distributing reservoir, a new intake, additional pumps, substantial strengthening of the distribution system and additional hydrants and gate valves; the grading then showed 344 points, an improvement of 700 points, or more than one class in the total grading of the city.

Another city in Ohio, with a population of about 55,000 showed 946 points of deficiency; improvements included additional pumps in a fireproof station, and substantial strengthening of the distribution system, and the new grading showed 383 points, an improvement of 563 points.

A city in New York, with a population of about 30,000 increased its supply, constructed an equalizing reservoir and greatly strengthened its distribution system; result, an improvement in grading of its water supply from 688 points to 94 points. This, together with improvements in the fire department and fire alarm give the city a low insurance base rate.

In a city in Pennsylvania, with a population exceeding 100,000, the superintendent of the water department showed a particularly keen interest in improving his system after a survey and report by the National Board of Fire Underwriters. Several conferences were held with the engineers of the Board, and practically all the recommendations made were complied with. The gates in a recently constructed dam were closed, giving a large additional impounding storage, two old pumps were replaced by a modern unit; steam, boiler feed and discharge piping were made more reliable; structural changes were made at the pumping station and additional fire protection provided; consumption was largely reduced by waste inspections; sub-

stantial improvements were made in the distribution system including the installation of four regulating valves; and numerous minor improvements. The result was a reduction in the grading from 340 points to less than 50 points, making the system one of the most adequate and reliable in the country from a fire protection standpoint. In this particular city there was no corresponding improvement in the other fire fighting facilities with the result that the net change in grading of the city was insufficient to change the class.

The cases above, which are only a few of those which might be cited, will indicate that while the water supply is only one of many factors in fire protection and prevention, it is a very important one, and one in which there are many opportunities for improvement. And let us suggest that when water supply improvements are being made the superintendent will do well to try and interest those in charge of other features of fire protection and prevention, so that they may keep pace with water supply and the maximum benefit in classification be obtained. The engineers of the National Board of Fire Underwriters and the bureau interested will be glad to cooperate and advise.

DISCUSSION

Malcolm Pirnie:[2] I think that an important lesson from this excellent paper was uttered in the last few summarizing sentences, mainly that we, as water works men, have a job beyond that of efficient operation of water works systems. There are several allied municipal services which must be improved along with water works to make fire protection in our communities effective.

When extensions, reinforcements or betterments of supply and pumping systems are planned we should coördinate the activities of civic bodies to improve the fire department, the police department, fire alarm systems, the general condition of pavements and streets in congested areas. We should strive for reasonable building codes which will gradually develop our cities into such bulwarks against fire that all cities will realize reductions in fire insurance rates to which so many are entitled.

There are a number of significant figures in the paper just presented to you which I will expand a little bit to give an idea of what water works men have already accomplished. In 1910 the average

[2] Consulting Engineer, New York, N. Y.

premium per hundred dollars on forty-three and one-tenth billion dollars worth of insured property was $1.077. In 1930, the average premium on one hundred and fifty-one and a third billion dollars worth of insured property was $0.769 cents per hundred, a reduction of 30.8 cents per hundred.

This means that today, through the services rendered and partly collected through water rates and through taxes (called today in current propaganda against taxes, "cost of government"), we are saving four hundred and sixty-eight million dollars a year over what we would have had to pay under the conditions that existed twenty years ago. That is a significant figure. It is part of a long list of figures that should appear on the other side of the balance sheet when talking about the cost of government.

There is another saving that water works men are particularly proud to have a considerable part in effecting and that is the extension in the life span which has been realized in the last twenty years. In this brief period, as compared with former decades, the life span has been extended practically ten years. Previous to 1906 it took eighty years to extend the life span ten years. The average life span has been extended in the last twenty years as much as in the eighty years preceding. Water purification and sterilization has been an important factor in this accomplishment during these last two decades when the rural population has been moving into the community centers. It is a significant achievement by water works men.

Saving in life can not be measured in dollars and cents. This saving is, partially due to water supply and also to increased knowledge of personal hygiene, better medical knowledge and care, and better waste disposal resulting from services created and maintained with tax money. But the purification of water supplies, the introduction and proper distribution of water in our communities, has been largely responsible for this extension of life. This has resulted in a reduction in life insurance in the last twenty years from $31.60 to $27.50 per thousand.

This saving per thousand dollars realized in the last two decades applied to the insurance in force in 1929, which amounted to one hundred and three billion dollars, results in an annual saving this year of four hundred and twenty-two million nine hundred thousand dollars. Adding the fire and life insurance savings together gives approximately nine hundred million dollars in these two insurance items alone. There are of course other savings that can be estimated in dollars which would add materially to this amount.

If we assume that water works are responsible for health protection in the proportion that they are rated to be responsible for fire protection (approximately one-third) we will obtain the amount contributed in dollars annually by water works through these two items alone. This amounts to three hundred million dollars a year, or something over 2 percent on the total investment that is in water works in this country. It seems to me that this is good talking information for us when we discuss the reasonableness of our demands that needed water works betterments should go forward.

This 2 percent is a constant extra dividend being declared each year that passes, to all water customers who already have benefited by receiving an indispensable service at cost which is worth much more than the bills rendered for it.

William W. Brush:[3] This paper gives a great deal of information that will be very useful and valuable. It has always seemed to me that the fire insurance group could be more definite in what allowances would be made when improvements are undertaken by a community and that more improvements would be undertaken that would be useful to the community if such more definite statements as to the change in rates were to be obtained, or made.

It is difficult to obtain from the officials that are in charge of the finances of any community the support for a program that makes for better fire protection which will ultimately be reflected in lower fire losses, and it is very hard if you have to say that the change will not make any difference as far as can be determined in the fire insurance rate. We appreciate to some degree, at least, the difficulties that are present in trying to determine what allowance should be made in fire insurance rates due to improvements for water supply. It does seem, however, as if the fire insurance interests should make a very special, determined and persistent effort to so arrange their rating methods that, when a community is apparently willing to undertake, improvements in its water supply for fire extinguishment purposes, a fairly definite indication will be given of the fire premium saving to the community from such an undertaking.

I trust that Mr. Booth and his associates will find some way whereby they can let us know these things and help us who are in the water works, and the fire extinguishment fields, by telling us that there will

[3] Chief Engineer, Department of Water Supply, Gas and Electricity, New York, N. Y.

be some lowering of the fire insurance rates if we undertake some proposed improvements which will definitely and undoubtedly lower ultimate fire losses.

PATRICK GEAR:[4] When we are out of subjects to talk on we can hit the insurance people, for they are well able to take care of themselves, but I sympathise with the Insurance Inspectors for they are checked and double checked. Our City is classed as No. 1, so they tell us. After spending $500,000 for improvements in the last five years we are still in class No. 1.

Mr. Pirnie says we could take some of these remarks home and have them published in our City papers, then the people would like to know where they come in on reduction of their insurance. Ask one of their insurance men here about it, and he will tell you that he does not make the rates, that it is up to another bureau, so they pass the buck along that way, and you just try and reach the fellow who makes the rates.

A few years ago we had a serious fire, and our fire department with the coöperation of the Springfield and Northampton departments who came 8 miles to our assistance, there was 27 streams of water playing on the fire at one time. We have a recording gage in our office which is about 1500 feet from the scene of the fire. The pressure on the gage indicated a drop of only 3 pounds, from 78 to 75 pounds and across the street from the fire the gage, at which we had a man stationed all during the fire, showed a drop of only 5 pounds.

The Insurance men were around the next day to investigate and came to my office. I told them all about the fire. I also showed them the pressure chart. Then they went back to make their report. One of their men returned to Holyoke a few days later. I asked him what the trouble was. He said that through some misinformation some of the Boston papers had said that the water pressure had failed at the Holyoke fire, he was back to check up on it. I told him here are the pressure charts take them back to Boston and see if they will be convinced.

I believe a city or town that is in class 1 should not waste any money to improve their system with the expectation of having their rates lowered, for you will not get it, but you will help the insurance company to make more money.

[4] Superintendent, Water Works, Holyoke, Mass.

LININGS FOR CAST IRON PIPE

By D. B. Stokes[1] and H. G. Reddick[2]

The principal reason why cast iron is lined is the prevention of tuberculation. We have prepared a map of the United States upon which is indicated areas where tuberculation is known to exist in varying degrees. This map was prepared on the basis of information obtained from a questionnaire sent to water works companies throughout the United States, by the Sectional Committee on Specifications for Cast Iron Pipe. We also give herewith the key for interpreting this map (fig. 1).

Key to map	*Source of supply*
x No tuberculation	R, River
o Moderate tuberculation	L, Lake
⊗ Severe tuberculation	W, Wells
	S, Springs
	I, Impounded

An examination of this map will show severe tuberculation in parts of Florida, Texas, South Carolina, eastern Pennsylvania and portions of New England. For instance, we find that at Charleston, South Carolina, the water supply is derived from a surface source with a low pH value and having a high organic content. Severe tuberculation took place at Charleston necessitating the cleaning of the distributing mains several times. Certain water treatments were carried out and J. E. Gibson, Manager and Engineer of the Charleston Water Department, concludes: "The Department feels that with the combination of the new method of treatment of the water and the cement lining of cast iron mains it has materially overcome its difficulties."

The prevention of tuberculation naturally implies the maintenance of carrying capacity and certain types of linings undoubtedly increase the carrying capacity over unlined pipe. In the Williams-Hazen formula, the value of the coefficient "C" for new cement lined pipe is

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approximately 145, whereas in certain pitch base linings the coefficient is increased to about 160.

Studies of tuberculation lead us to believe that exposed iron is a necessary condition to the formation of tubercles. This belief is concurred in abroad. Messrs. Fred W. Hammond and Arthur Goffey, writing in Water and Water Engineering, January 20, 1932, state: "It is universally found here that for the growth of definite tubercules of ferric hydroxide the presence of exposed iron is essential, the smallest pin hole in the protective coating being sufficient to provide a nucleus."

Geographical Distribution of Tuberculation in Water Pipe

FIG. 1

One of the early materials used to line the inside of water pipe was cement. Some 90 years ago the French Academy of Science reported favorably on the use of cement to prevent tuberculation, and it has been employed for that purpose with varying degrees of success up to the present time.

Its use, however, has encountered several difficulties. It may become loose from the pipe or it may crack and expose the iron. Another very serious feature is that, when first installed, the water passing through the pipe is very hard and many complaints arise from households and industrial establishments.

It was considered that the hardness of the water was due to the

leaching of the lime caused by the breaking down of tricalcium silicate into dicalcium silicate and free lime. Accordingly, experiments were instituted to eliminate the free lime in cement and it was ascertained that by mixing with the cement mortar certain proportions of active silicious materials, such as some blast furnace slags and phospheric acid, the free lime was practically eliminated. At the end of 24 hours, analyses showed 0.3 to 0.4 percent free lime, and at the end of 48 hours, no free lime was found.

Without the treatment with phosphoric acid and finely ground silicious aggregate, the free lime at the end of 24 hours was from 3 to 6 percent depending upon the nature of the cement.

Leaching tests were then instituted as follows: A 12-inch pipe 2-feet long was lined with the cement mixture under observation. After capping one end with non-leachable material, the pipe was filled with distilled water and a stainless steel plunger kept the water continually agitated for five weeks, a sample of the water being analyzed at the end of each successive week. The analyses showed that, although the free lime had been eliminated, the hardness of the water had only been decreased by a little over one half. In other words free lime is not the sole agent which causes the water to be hard. The alkalies also leach out and some of the aluminum compounds hydrolyze.

Effort was next directed toward minimizing the leaching of alkali and alumina compounds. It was at first thought that if a hard gloss could be put on the inner surface of a cement lined pipe, the leaching trouble would be alleviated.

When applying a cement lining centrifugally, this gloss can be put on by maintaining a proper water ratio together with spinning time and spinning rate. This was done but, unfortunately, the laitance or "slurry" which was thus brought to the surface was very high in alkali and alumina content and naturally would leach away very early in a pipe line and would make the water extremely nauseating and hard in the first days after installation. This was shown again by the leaching tests. The water was bitter and alkaline to taste and to litmus and was absolutely unpotable.

This, however, shows one thing very strikingly; if the pipe user insists on a smooth interior he will receive a product which, in the early days of service, will give a very hard water. If, on the other hand, a proper water ratio and spinning is allowed to bring this laitance to the surface and it is poured off, then a material reduction

in this objectionable leaching will be accomplished, but the surface will not be so smooth.

The above remarks relative to the smoothness of linings are applicable when they are made in accordance with the American Water Works Association specifications. Smoothness of cement linings is undoubtedly desirable and several experimental mixes are now being worked on which give the desired smoothness without the usual resulting water hardness. In order to obtain an even, dense and homogeneous protective coating cement linings are applied centrifugally whenever possible.

The difficulties encountered, due to looseness and cracking of linings, are also well known to manufacturers and to many consumers.

DIFFERENTIAL TEMPERATURE STRESSES

The application of cement linings to small diameter pipe does not present serious difficulties, but in 20-inch diameter pipe and larger, the problem is not so simple. Although the coefficient of expansion of cement and cast iron in normal temperature ranges may be approximately the same, this does not solve the problem of possible looseness and cracking. For, if a cement lined pipe is subjected to the sun's rays, the iron, particularly if not covered with a white substance, such as whitewash, on the exterior, will heat up much more rapidly than the cement lining and set up strains between the two. If the bond between the lining and the pipe is not correctly proportioned with the strength of cement, loose spots, cracking, or possibly both, will occur. If the bond and strength are properly proportioned, checking or crazing of the lining may result but no serious looseness will be encountered.

It should be emphasized that it is not so much the natural shrinkage of the cement, upon setting, which causes looseness and cracking, as it is the differential expansion of the iron and the cement after the cement is set. In support of this contention, it can be pointed out that much less trouble occurs with cement lining in moderate weather than in hot summer weather, whereas the natural shrinkage of the cement does not vary under these conditions. We must, therefore, look for some other cause for looseness and cracks than the normal volume changes which occur in cement upon hydration.

We have found in a temperate climate that the difference in temperature, between a black pipe and one protected on the exterior with white wash, will amount to as much as thirty degrees when the

pipes are exposed to the direct rays of the sun. For that reason, all of our larger sizes of cement lined pipe are now protected on the exterior with a whitewash coating before shipment.

Temperatures of the lining have been ascertained as well as the temperature of pipe and in normal summer weather there is a great difference between the two. One cannot, therefore, escape the fact that when a cement lined pipe is exposed to temperature changes, there will exist strains between the lining and the pipe, which must result in one of two conditions, if they are of sufficient magnitude. These two conditions are crazing or checking of the lining, with little or no looseness, or decided looseness usually accompanied with cracks of considerable magnitude. It is obvious that after the pipe is laid and in service, the conditions discussed above would normally not exist. The major troubles with cement lining as far as cracking and looseness are concerned are, therefore, to be expected in hot weather, during the time that the pipe is in storage or transit.

It is not our intention to overemphasize the importance of the conditions just described. It has been definitely determined, for example, that loose linings, after the pipe has been filled with water, expand against the side of the pipe. Likewise, we have no evidence that the utility of the main or lining is appreciably lowered, due to the presence of cracks or crazing of normal magnitude.

Admitting, therefore, the several objectionable conditions that may exist in cement lining made under our best modern practices, we still are of the opinion that cement lining has done a great deal to eliminate the troubles from tuberculation and do not hesitate to predict further developments of great interest and benefit to the water works engineer, particularly along the lines of a non-leaching cement.

COATING THE CEMENT

Another method of reducing the leaching action of water when passing through the cement lined mains is to paint or spray the cement lining with some medium generally bituminous in character. This, however, must be approached very circumspectly. You cannot apply at random any spray such as might be done in road work.

In the first place the paint or spray used must impart no odor, taste, or color to the water. In the second place it should not be applied until after 48 hours of curing in an atmosphere of approximately 100 percent relative humidity. If applied in this manner the

paint will aid in the final curing of the cement and decrease, materially, the early leaching.

Cement lined pipe have been heated and dipped in hot tar, but this procedure manifests itself in very serious checking and loosening of the lining. This may not be noticed at the time but after the tar is dry it may readily be seen.

BITUMASTIC LININGS

Bitumastic linings, as now supplied by the United States Pipe and Foundry Company, consist of a pitch base combined with a proper mineral filler in order to give the desired physical properties. Such a lining cannot leach and, when applied centrifugally, gives an exceedingly smooth and brilliant surface which, experience shows, prevents the lodgment of Algae and similar growths. This type of lining is preferred in some sections for this reason. The difficulties of this lining consist in troubles encountered during storage and transit. If the lining is so proportioned to resist the effects of cold weather, it will flow when subjected to summer temperature. If properly proportioned for summer conditions, it will become brittle and crack under conditions of extreme cold. Present practice is, therefore, to proportion the material with the idea of laying the pipe as soon as possible after the lining operation, in order to prevent its being subjected to a wide range of temperature variation. At the present time this type of lining does not present a material which can be applied and left on the yard during the winter and summer months without difficulties being encountered.

PITCH BASE LINING

Another lining which completely covers the iron and prevents tuberculation and when applied centrifugally gives such a smooth surface that the lodgment of Algae and slimy streamers is next to impossible, is a pitch base lining, now being experimented with to overcome the objections of Bitumastic lining. Pitch was chosen as a base for this lining material because it imparts no color, odor or taste to water.

One of the difficulties with mastics, which have been in use, is their inability to withstand the climatic range of temperature as previously explained. If suitable for summer application they become brittle and crack in cold weather. If suitable for winter use they wrinkle and run at summer temperatures. Another difficulty

has been the mineral filler which, during centrifugal application, segregates to the pipe wall and materially decreases the bond between the pipe and lining and, at times, results in serious cracking and falling away from the pipe.

Starting with pitch as a base the problem is then to make this brittle material, with too low a softening point, into a plastic, tough material with a sufficiently high melting point.

The mode of attack was to plasticize the pitch. This was brought about by adding to the pitch a small percentage of a plasticizing agent. Under proper methods of manufacture, this produces a material with sufficient plasticity to adhere firmly without cracking or flaking at zero degrees Fahrenheit and of sufficient rigidity, without the use of a filler, to withstand temperatures of 170°F. and more. This material, when applied to a pipe centrifugally, will withstand the summer sun and the winter cold and produces a very smooth and glossy lining, free from bubbles and pits. On account of its complete covering power and extreme smoothness, it is unfavorable to tuberculation or the lodgment of plankton of any kind. Naturally the normal efficiency of the line will be maintained.

ASPHALT LINING

One naturally cannot discuss a plastic lining without saying a word about asphalt linings, which have been used quite extensively abroad. These asphalt linings have been loaded with lime and other fillers, some of which we have tried. We also loaded the asphalt with cement and had excellent results, with the exception of the fact that as far as we have gone we have always found, under our leaching conditions, odor, taste and color.

VITREOUS ENAMEL

One more lining, which we wish to discuss, is a vitreous enamel. As is well known, enamel may be made with any coefficient of expansion suitable to iron or steel. Very excellent looking linings have been made with vitreous enamel, but we are not prepared to say at the present time that they will entirely prevent tuberculation. Although such enamels can be made to adhere very tenaciously to the surface to which they are applied, they will not undergo deformations greater than the elastic limit of the enamel itself without resulting in checks, cracks and looseness. The ductility of the material entering into the fabrication of the pipe itself is not of particular importance when

vitreous enamels are applied, as the amount of deformation is limited by the possible movement of the enamel itself. Such linings do have a very smooth and glossy surface with a resulting low coefficient of friction and, consequently, a high carrying capacity. Furthermore, the lining is very thin and the effective diameter of the pipe is not materially reduced by its application.

FRICTION COEFFICIENT TESTS ON CAST IRON DISTRIBUTION MAINS WITH CENTRIFUGALLY APPLIED BITUMASTIC ENAMEL LINING

By Elson T. Killam[1]

Development of the centrifugal process for the application of cast iron pipe lining, has resulted in the attainment of interior surfaces which are obviously so smooth, that attention is at once directed to the probable value of the friction coefficient.

The increase in the adoption of the so-called Bitumastic Enamel lining, centrifugally applied, has to-date been largely predicated upon the apparent smoothness of the lining, rather than upon any available coefficient data. This condition arises because the material has been available, on an economical production basis, for only a brief period, and consequently there are no published data, at least within the knowledge of the writer, upon the value of the friction coefficient with this type of lining.

The controlling effect of the friction coefficient in the selection of pipe diameters, is generally recognized, and accordingly, it is most important that definite information upon the value of the coefficient, with various types of linings, be made generally available.

It is with this thought in mind that the following described series of tests were carried out.

To briefly state the results of these tests, it has been found that the value of the friction coefficient "C" in the Williams-Hazen formula, for pipe lined with this material, is 150.

This result is based on nominal diameters, and also upon actual conditions prevailing in a distribution system, and accordingly this value includes the various minor losses due to bends, tees, and to other similar disturbances to flow. The results of these tests are reported in terms of the Williams-Hazen coefficient in the formula $V = cr^{0.63}\ s^{0.54}\ 0.001^{-0.04}$.

It is pertinent to emphasize the fact that for a fixed friction loss, or loss of pressure due to a certain flow through a pipe line, the relative carrying capacity is directly proportional to the friction

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coefficient. In other words, a pipe with a coefficient of "C" = 150, will have 150 percent of the carrying capacity of a pipe with a coefficient of "C" = 100.

The tests described herein were made immediately after the completion of a new distribution system, and, therefore, the coefficient values obtained represent initial values only. The record of the material used for the lining, however, has been such as to indicate that the initial high carrying capacity may prove permanent.

The economic importance of such a possibility is so apparent, that it is earnestly to be desired that steps will be taken by all who are able so to do, not only to secure additional information about the initial coefficient, but also to arrange to check the coefficient value from time to time over a substantial period of years.

It is the hope of the writer that the description of the test methods and the results presented in this paper, will serve to focus more attention upon this subject, and thereby interest others in carrying out similar tests.

SUMMARY OF RESULTS

The tests indicate that the value of the Williams-Hazen coefficient in a new distribution system lined with Bitumastic Enamel, centrifugally applied, is equal to "C" = 150.

GENERAL METHOD OF CONDUCTING TESTS

In carrying out the tests, sections of the distribution system were isolated by closing the necessary valves. The intermediate service reservoir, a reinforced concrete tank 62 feet in diameter and 23.5 feet in height, together with the main 12-inch supply line leading from this tank, were used in all of the tests.

Draft from the tank was maintained through the test section, by opening one or more hydrants beyond and downstream from the downstream pressure station.

The rate of flow was determined by measuring the volume drawn from the tank.

The elevation of the energy gradient, at the upstream pressure station, was derived from the water level in the tank, and at the downstream pressure stations was obtained from pressure gages set at known elevations.

In order to develop as high a total loss of head as possible, and thereby reduce the effect of errors in measurements to a minimum, a

line of the greatest feasible length was selected, and a draft of the highest possible rate was maintained.

CONTROLLING FEATURES OF TESTS

1. Location and general conditions

The tests were conducted in Parsippany-Troy Hills Township, Morris County, New Jersey, about thirty miles west of New York City.

The entire distribution system, comprising 25 miles of centrifugally spun pipe in 16-foot lengths, and from 6 to 12 inches in diameter, was lined with this material. Inasmuch as no taps had been made for

FIG. 1. CAST IRON PIPE LINED WITH BITUMASTIC ENAMEL

house services, complications arising due to draft from a section under test, were not a factor in these tests.

2. Tests represent actual field conditions

The tests were conducted on a portion of a newly constructed distribution system, and not on a pipe line laid especially for test purposes.

This matter is of considerable significance because of the fact that, in the majority of cases where coefficient tests have been undertaken, the tests were generally conducted on supply lines, or on pipe lines laid particularly for the purpose of conducting such tests.

Obviously, the coefficient values reported herein, will be lower than those found on a supply line or on a test line, because under the conditions prevailing for the tests herein described, there were many incidental losses due to bends, curvature in alignment, hydrant tees, valves, etc., and these losses were absorbed by the coefficients as reported.

3. Coefficients reported based on nominal diameter

The reported coefficients were based on nominal diameters, this method being adopted after some consideration, as being more practical, because of the relatively small deviation from true nominal diameters.

It is logical to expect that the actual net diameter would in all cases be less than the nominal diameter, because of the reduction of diameter due to the lining material, which in this instance had a specified minimum thickness of three-thirty seconds of an inch ($\frac{3}{32}''$).

The cast iron pipe in this system, however, was centrifugally spun pipe, and the diameter of this type of pipe is ordinarily greater than nominal, the average excess diameter for 6- to 12-inch sizes, for this type and class, being from 0.2 to 0.3 inch.

A considerable number of lined pipe were calipered, and it was found that the reduction of two times the thickness of the lining ($\frac{3}{16}''$) was compensated for by the oversize pipe, so that the resulting diameter of the lined pipe was in all cases practically equivalent to the nominal diameter.

4. Length and size of lines tested

The line in Test No. 1 consisted of 5,271 linear feet of 12-inch pipe. The line in Test No. 2 had a total length of 17,754 linear feet, of which 5,490 feet were 12 inches in diameter, and the balance of 12,264 feet were 10 inches in diameter.

In Test No. 2, an intermediate reading was taken between the upstream and downstream pressure stations, and the results were reported separately on the first stretch, composed of 5,490 feet of 12-inch pipe and 2,857 feet of 10-inch pipe, and upon the second stretch, consisting of 9,407 feet of 10-inch pipe. Computations were also made upon the overall length of the line 17,754 feet, for which the upstream and downstream pressure readings were used, the readings at the intermediate station being neglected.

5. Methods of computation

All hydraulic computations were made with the Williams-Hazen slide rule, the results being reported as the value of the Williams-Hazen coefficient "C" in the formula $V = cr^{0.63}\ s^{0.54}\ 0.001^{-0.04}$.

6. Allowances for minor losses

The only allowance for minor head loss not absorbed by the coefficient, was the loss of entry at the tank.

In determining the elevation of the energy gradient at the upstream pressure station, the entry loss was assumed to be $0.5\ \frac{V^2}{2g}$, and the proper head was deducted from the elevation of the water level in the tank.

7. Correction for leakage found unnecessary

Leakage tests conducted prior to the coefficient tests showed an average rate of leakage, under a test pressure of 150 pounds per square inch, of 82 gallons per inch-mile per day, or the equivalent of 2.04 gallons per minute total leakage for the longest section tested.

Inasmuch as the rate of flow maintained during the test varied from a minimum of 1,025 to a maximum of 2,585 g.p.m., it is obvious that this leakage would have no practical effect on the results, within the limitations of possible accuracy.

8. Measurements required to determine value of coefficient "C"

In order to obtain the value of coefficient "C," the following factors in the formula $V = cr^{0.63}\ s^{0.54}\ 0.001^{-0.04}$, were necessarily determined:

V = the velocity in the line, in feet per second.
r = the hydraulic radius of the pipe, in feet.
S = the slope of the hydraulic gradient, in feet per thousand.

The term $0.001^{-0.04}$ is a constant, and therefore requires the determination of no data, this constant being adopted in the Williams-Hazen formula, in order to make the value of "C" comparable with the value of "C" in the older Chezy formula.

9. Methods of measurement

V = velocity in feet per second

The velocity was obtained by dividing the quantity flowing, by the cross sectional area of the pipe, based on nominal diameter.

The rates of flow were computed from measurement of rate of drop of water surface in the concrete distribution storage tank, these computed rates being checked in some instances by a hydrant pitot, placed on hydrants discharging the test flow below the downstream pressure station.

The drop of water level in the tank was measured at one minute intervals; during a portion of the tests by means of a hook gage, and during the balance of the tests by a leveling rod.

It is to be noted that comparatively high velocities, ranging from a minimum of 2.91 to a maximum of 7.35 feet per second, were developed during the tests, and it is believed that this condition was an important factor in obtaining reasonable accuracy of results.

S = *slope in feet per thousand*

The loss of head between pressure stations required coincident readings of the elevation of the gradient at both stations.

The gradient elevation at the upstream station was determined from observations of the water level in the tank, taken at one-minute intervals throughout the tests.

The actual elevation of the energy gradient at the upstream station was assumed to be the elevation of the water level in the tank, minus $0.5 \frac{V^2}{2g}$, this latter amount representing the assumed entrance loss into the pipe line leading from the tank.

The elevation of the gradient at the downstream pressure station was obtained from duplicate pressure gages set at known elevations. The average gage readings, transferred into equivalent feet of water, were added to the elevation of the gage centerline, and the velocity head in feet was added to this sum, in order to obtain the actual elevation of the energy gradient.

The connections to the gages were provided with blow-offs, and during the interval between readings, a slight flow was maintained, in order to make certain that the riser to the gage would be free from air.

Before beginning the tests, bench levels and check levels were run between each of the pressure stations.

The length of line between the various stations was obtained by measuring along the surface of the ground with a steel tape, the distances thus obtained being checked by comparison with the stationing established for the final estimate and for record plans of the system.

The slope of the hydraulic gradient, or friction loss, was obtained by dividing the total loss of head between upstream and downstream pressure stations, by the distance between the respective stations.

10. Consistency of results under varying conditions

Examination of the accompanying tabulations summarizing the readings and results of the tests, indicates a reasonable uniformity of values of the coefficient, with velocities varying from 2.91 to 7.35 feet per second, with diameters of 10 and 12 inches, and with a section composed of both 10 and 12 inch pipe.

It was also found that the arithmetical average value of the coefficient, derived from separate computations for each ten-minute period, was practically the same as the value derived from the average rate of flow and the average gradient maintained throughout each test from beginning to end.

It was also found in Test No. 2, that the results with the entire section, 17,754 feet long, were practically the same as the results in the two adjacent shorter sections making up the total section.

11. Probable range of accuracy

The probable range of accuracy is indicated in table 1 which summarizes the effect which certain assumed errors in measurement would have, on the coefficient value.

In all cases, the errors are assumed well beyond the range of any probable error, in order to set up extreme limits.

For instance, although it is assumed in this tabulation that the gage reading was two pounds too high or too low, it is believed that actual conditions lie well within this range, because of the fact that the gages were in duplicate, were calibrated both before and after the tests, and particular care was taken to maintain the connections free from air.

An assumption that the tank diameter was one foot greater or smaller than that shown on the plans, is obviously a major exaggeration, because of the fact that the diameter was carefully checked with a steel tape, and in no case was there found a deviation of more than 0.03 feet from the diameter shown on the plans.

An error of 0.04 feet in measurement of the water surface in the tank, also was considerably beyond the range of a possible error with a hook gage, but was a possible condition when the measurements were taken with a rod. Any error in determination of the water level,

however, would be neither constant nor accumulative, but would be compensating. This condition exists because of the fact that if the measured drop in one period of a minute was too great, then the excess would in all probability be taken up in the subsequent reading,

TABLE 1

Effect of certain assumed errors in measurements on value of Williams-Hazen coefficient "C"

(Assumed errors in all cases beyond probable limits)

Test No. 2, April 10, 1932

+ denotes reported value would be too high. − denotes reported value would be too low.

CAUSE OF ASSUMED ERROR	DATE OF TEST	EFFECT ON W. & H. COEFFICIENT "C"		
		At maximum test flow time reading 5:04	At average test flow time reading 5:24	At minimum test flow time reading 6:14
1. Gage reading 2 lbs. too high	April 10	+1	+4	+6
2. Gage reading 2 lbs. too low	April 10	−4	−6	−7
3. Tank diameter 1 foot more than plans	April 10	−4	−5	−4
4. Tank diameter 1 foot less than plans	April 10	+4	+4	+4
5. 10 min. drop in Water Level 0.04′ less than measured*	April 10	+10	+9	+12
6. 10 min. drop in Water Level 0.04′ more than measured*	April 10	−8	−10	−12
7. Errors in levelling between pressure Stations	April 10	Negligible	Negligible	Negligible

* This error would be compensating and not constant or cumulative.

and the averaging of both measurements would tend to balance the error.

12. Considerations essential to obtaining satisfactory test results

In order to obtain satisfactory results from tests of this nature, careful consideration must be given to the following matters.

(a) It is essential to select a strategically located section of line

for the test, in order that the hydraulic conditions occurring during the test may be favorable for the various measurements.

(b) It is essential to work through the test in the office, in advance of the actual field test, in order to make certain that adequate rates of flow can be developed, and that the gradients will lie within suitable measuring range, for whatever device may be selected for measurement of pressure.

(c) It is desirable to create as great a total loss of head as feasible, and to utilize as long a length of pipe line as possible, in order to assure accuracy of results. The development of high rates of flow may obviously cause considerable difficulty in discharging the water, when the pipe line has a large diameter.

(d) It is essential to determine all basic data, such as the distance between stations, and the relative elevation of gages, in advance of the test, in order to compute results on the ground during the test. By so doing, any unusual condition will be at once evident, and can be corrected before the completion of the test.

13. Personnel

The tests were planned and carried out under the direction of the writer, assisted by Messrs. Brown, Carlson, Huang, Klegerman, Portal, Ralston, Roberts, Schetne, Sloboda, Wilson, Wold and Wood; members of the staff of Alexander Potter.

The meter used in Test No. 3, and various pipe fittings, were made available through the cooperation of the Water Department of the City of Newark, New Jersey.

14. Diagrams and tables

There are appended hereto, various diagrams illustrating the test methods and the results, and inasmuch as these diagrams are practically self-explanatory, descriptions of the same are limited to a brief statement of the subject covered by each diagram.

Figure 2 represents the opposing trend between capacity demand, and available capacity of a pipe line. Obviously, the diagram represents a hypothetical case only, and is presented only to illustrate the economic importance of providing permanent carrying capacity of a pipe line. The slope of both lines depends largely on local conditions, but the opposing slope of the lines represents an economic waste which is present in every water system.

Figure 3 illustrates the features of Test No. 1, together with a summary of results.

Figure 4 represents the features and results of Test No. 2.

Tables 2 and 3 show the various observations and the various values of the coefficient obtained in both test No. 1 and test No. 2.

Figures 5 and 6 illustrate a test not previously mentioned in the paper, which was conducted in November, 1931, and which represents the initial attempt to determine the coefficient.

The results of this test were not satisfactory, and these diagrams

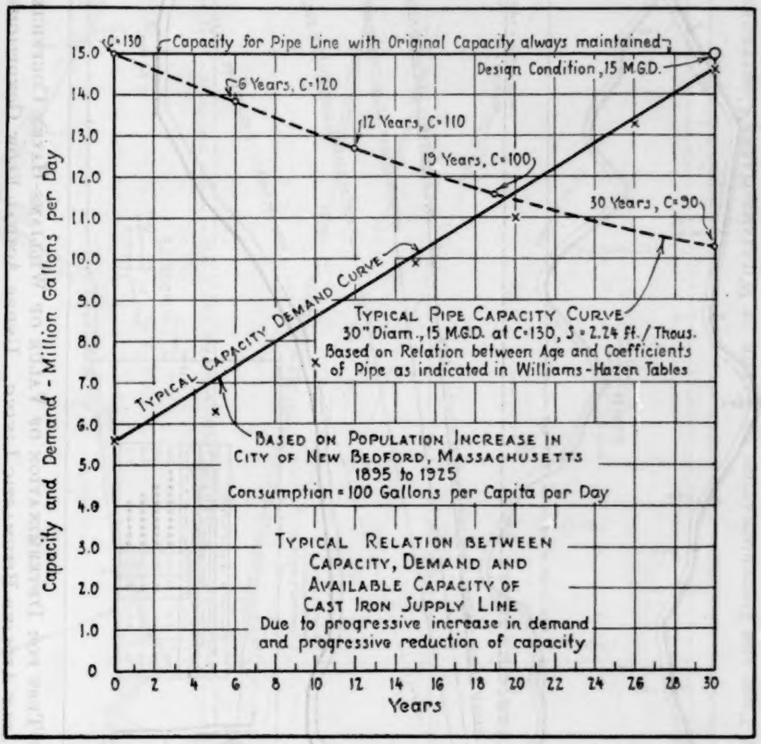

FIG. 2

are presented only to illustrate an alternative method of conducting a test, and to point out possible difficulties which may be encountered.

In this test, flow through the test section was maintained by direct pumping, and with no storage on the line. Discharge was through a hydrant, as in the case of the later tests.

The elevation of the pressure gradient was determined from the free surface of water in water columns at the upstream and down-

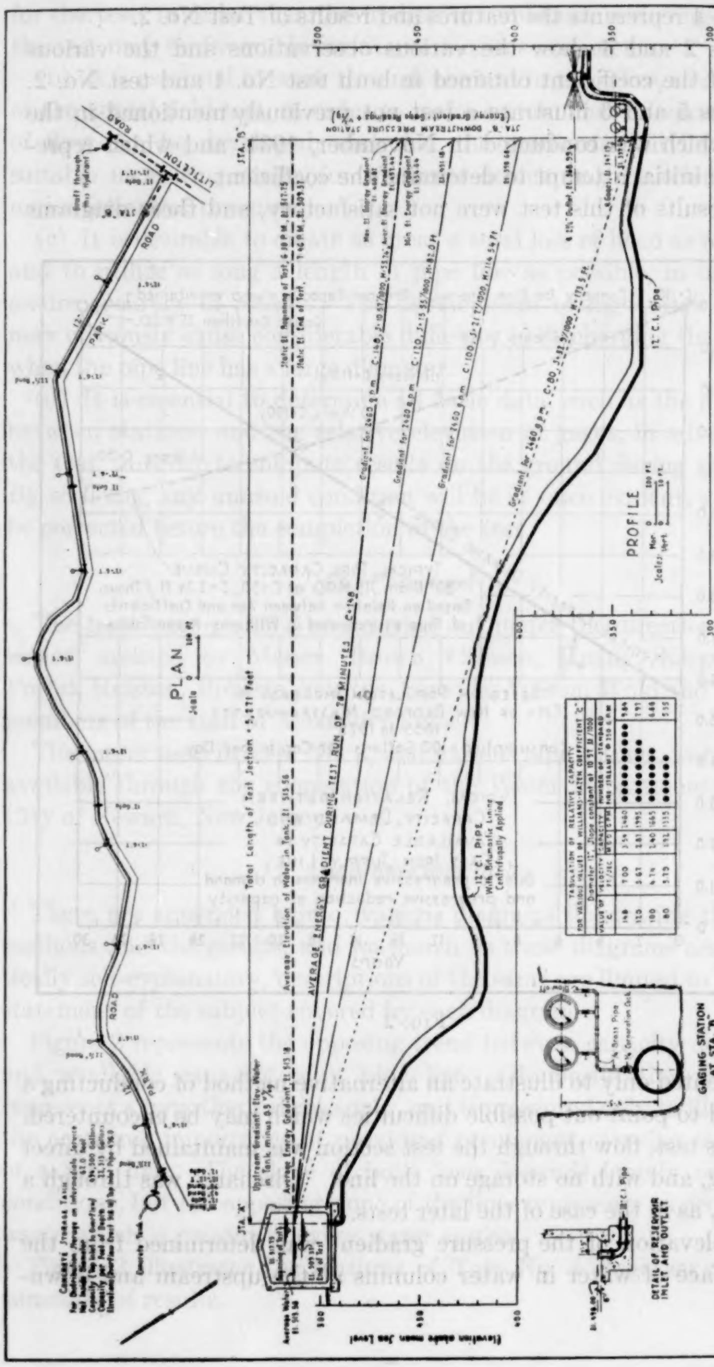

FIG. 3. CONTROLLING FEATURES OF TEST FOR DETERMINATION OF VALUE OF WILLIAMS-HAZEN COEFFICIENT "C" FOR CAST IRON PIPE LINED WITH CENTRIFUGALLY APPLIED BITUMASTIC LINING, UNDER ACTUAL FLOW CONDITIONS IN DISTRIBUTION SYSTEM, APRIL 9, 1932

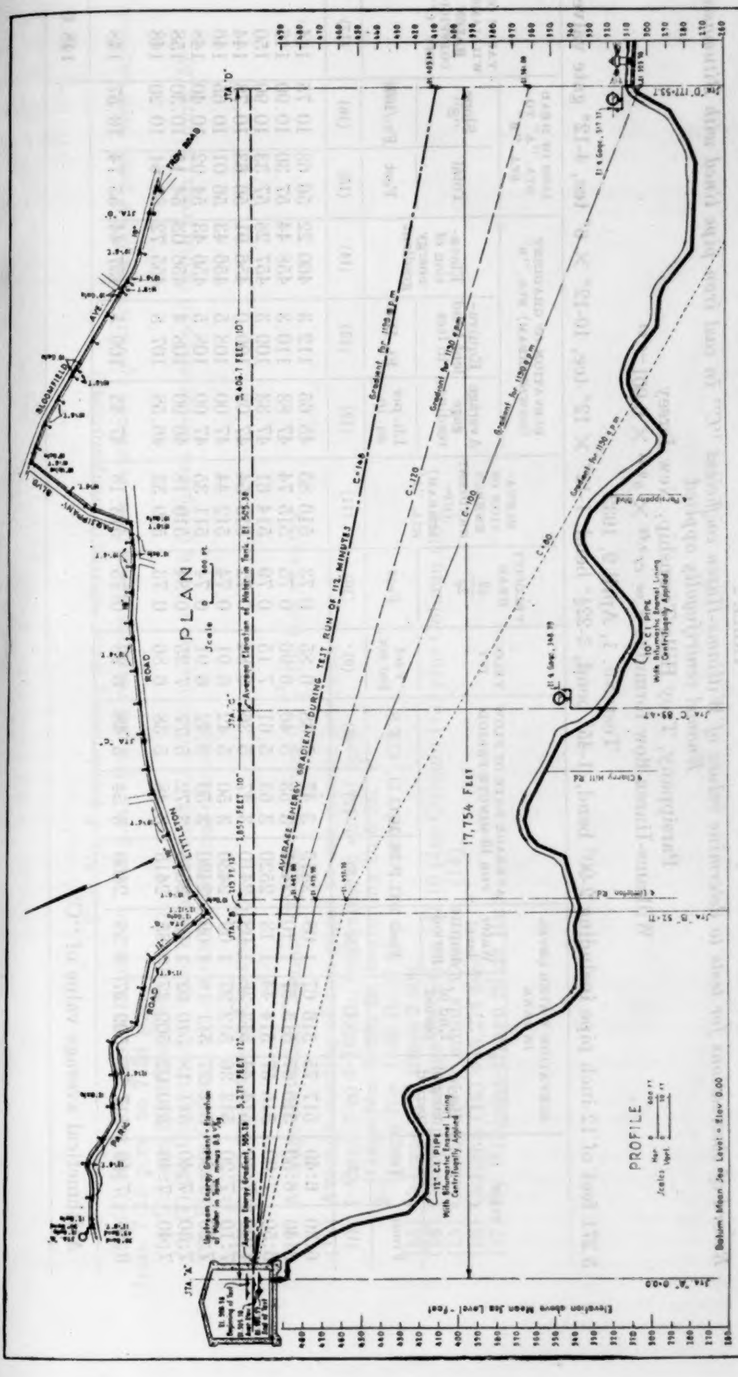

FIG. 4. CONTROLLING FEATURES OF TEST FOR DETERMINATION OF VALUE OF WILLIAMS-HAZEN COEFFICIENT "C" FOR CAST IRON PIPE LINED WITH CENTRIFUGALLY APPLIED BITUMASTIC LINING. UNDER ACTUAL FLOW CONDITIONS IN DISTRIBUTION SYSTEM, APRIL 10, 1932

TABLE 2

Record of observations for tests to determine values of Williams-Hazen coefficient "C" in cast iron pipe lined with Bitumastic Enamel centrifugally applied

Parsippany, Troy Hills Township, New Jersey

Williams-Hazen flow formula: $v = cr^{0.63} \times s^{0.54} \times 0.001^{-0.04}$

Test No. 1, April 9, 1932

5,271 feet of 12-inch pipe including 2-90° bend, 1-45° bend, 3-22½° bend, 1-12″ × 12″ tee, 10-12″ × 6″ tee, 4-12″ gate valves

TIME		ELEVATION WATER LEVEL IN TANK			AVERAGE RATE OF FLOW FOR 10-MINUTE PERIOD			VELOCITY	VELOCITY HEAD $\frac{v^2}{2g}$	ELEVATION OF ENERGY GRADIENT (UPSTREAM) STA. "A"	ELEVATION OF GRADIENT (DOWNSTREAM) STA. "B"			LOSS OF HEAD STA. "A" TO STA. "B"		VALUE OF WILLIAMS-HAZEN COEFFICIENT "C"
From	To	Beginning of period	End of period	Water level drop in period							Average gage reading	Equivalent head in feet	Elevation of energy gradient	Total	Slope "S"	
				Feet	G.P.M.	M.G.D.	C.F.S.	Feet per sec.	Feet		Lb. per sq. in.	Ft. H_2O		Feet	Ft./1000	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
6:30	6:40	517.75	516.67	1.08	2415	3.47	5.38	6.85	0.73	516.85	48.65	112.3	460.22	56.63	10.75	144
6:40	6:50	516.67	515.57	1.10	2455	3.53	5.46	6.96	0.75	515.74	47.83	110.3	458.44	57.30	10.90	146
6:50	7:00	515.57	514.44	1.13	2520	3.63	5.61	7.15	0.79	514.61	47.33	109.3	457.28	57.33	10.90	150
7:00	7:10	514.44	513.36	1.08	2410	3.47	5.36	6.84	0.72	513.54	47.08	109.0	456.91	56.63	10.78	144
7:10	7:20	513.36	512.27	1.09	2430	3.50	5.42	6.91	0.74	512.44	47.00	108.5	456.43	56.01	10.65	146
7:20	7:30	512.27	511.18	1.09	2430	3.50	5.42	6.91	0.74	511.35	47.00	108.5	456.43	54.92	10.40	148
7:30	7:40	511.18	510.02	1.16	2585	3.72	5.77	7.35	0.84	510.18	46.90	108.4	456.03	54.15	10.30	158
7:40	7:46	510.02	509.37	0.65	2415	3.48	5.38	6.86	0.73	509.33	46.58	107.8	455.72	53.61	10.20	148
6:30	7:46	517.75	509.37	8.38	2460	3.54	5.48	6.98	0.75	513.18	47.45	109.5	457.44	55.74	10.57	148
Arithmetical average value of "C"																148.0

Test No. 1 (explanatory notes relative to computations)

(3) (4) By hook gage 6:15 p.m. to 7:12 p.m. By rod readings down from reference elevation on hook gage pipe support from 7:13 p.m. to 7:54 p.m.

(5) Column (4) minus Column (3).

(6) (7) (8) Computed on basis of checked net plan dimensions of tank (columns deducted) and depth from Column (5).

(9) Flow in Column (6) ÷ area for nominal diameter.

(10) $\frac{v^2}{2g}$ for velocities in Column (9).

(11) Average elevation Columns (3) and (4) minus $0.5\frac{v^2}{2g}$ for loss at entry.

(12) Average reading of two gages at one minute intervals
1–Gage No. 4–160 lb. beginning to end of test.
2–{Gage No. 3–200 lb. beginning to 6:36.
Gage No. 1–50 lb. 6:38 p.m. to end.

(13) Column (12) times 2.308.

(14) Center line-gage elevation 347.19 plus Column (13) plus Column (10).

(15) Column (11) minus Column (14).

(16) Column (15) ÷ 5,271 feet.

(17) By Williams-Hazen Slide Rule.

TABLE 3

Record of observations for tests to determine values of Williams-Hazen coefficient "C" in cas

Parsippany, Troy Hills Township, N

Williams-Hazen flow formula: $v = cr^{0.63} \times$

5,490 feet of 12-inch pipe, 12,263.7 feet of 10-inch pipe, 3-10″-22½° bend, 3-12″-22½° bend, 27-10

TIME		ELEVATION WATER LEVEL IN TANK			AVERAGE RATE OF FLOW FOR 10-MINUTE PERIOD			VELOCITY FEET PER SEC.		VELOCITY HEAD IN FEET		DATA STATION "A" TO STATION "C"							
												Elevation of energy gradient (upstream) Sta. "A"	Elevation of gradient (downstream) Sta. "C"			Loss of head Sta. "A" to Sta. "C"		Value of Williams-Hazen coefficient "C"	Elevation of energy gradient (upstream)
From	To	Beginning of period	End of period	Water level drop in period				In 10″ ϕ	In 12″ ϕ	In 10″ ϕ	In 12″ ϕ		Gage reading	Equivalent head	Elevation of energy gradient	Total	Slope "S"		
				Ft.	G.P.M.	M.G.D.	C.F.S.	Ft./sec.	Ft./sec.	Feet	Feet		Lb. per sq. in.	Ft. H_2O		Feet	Ft./1000		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
4:54	5:04	508.45	507.77	0.68	1520	2.19	3.39	6.27	4.32	0.61	0.29	507.96	47.50	110.0	459.36	48.60	9.48	156	459.3
5:04	5:14	507.77	507.18	0.59	1320	1.90	2.94	5.44	3.75	0.46	0.22	507.37	50.30	116.2	465.41	41.96	8.19	147	465.4
5:14	5:24	507.18	506.58	0.60	1340	1.93	2.99	5.50	3.80	0.47	0.22	506.77	50.50	116.7	465.92	40.85	7.98	151	465.9
5:24	5:34	506.58	506.00	0.58	1295	1.86	2.89	5.35	3.70	0.44	0.21	506.18	51.65	119.0	468.19	37.99	7.43	151	468.
5:34	5:44	506.00	505.48	0.52	1160	1.67	2.58	4.75	3.30	0.35	0.17	505.66	53.00	122.5	471.60	34.06	6.66	145	471.
5:44	5:54	505.48	504.97	0.51	1140	1.64	2.54	4.70	3.23	0.34	0.16	505.14	55.00	127.0	476.09	29.05	5.68	154	476.
5:54	6:04	504.97	504.46	0.51	1140	1.64	2.54	4.70	3.23	0.34	0.16	504.63	55.25	127.8	476.89	27.74	5.42	156	476.
6:04	6:14	504.46	504.00	0.46	1025	1.48	2.28	4.20	2.91	0.27	0.13	504.16	55.25	127.3	476.32	27.84	5.46	143	476.
6:14	6:24	504.00	503.54	0.46	1025	1.48	2.28	4.20	2.91	0.27	0.13	503.70	55.05	127.0	476.02	27.68	5.41	144	476.
6:24	6:34	503.54	503.05	0.49	1095	1.58	2.45	4.50	3.12	0.31	0.15	503.23	55.00	127.0	476.06	27.17	5.31	155	476.
6:34	6:44	503.05	502.54	0.51	1140	1.64	2.54	4.70	3.22	0.34	0.16	502.71	54.95	126.9	475.99	26.72	5.22	162	475.
6:44	6:47	502.54	502.39	0.15	1120	1.61	2.50	4.62	3.17	0.33	0.16	502.38	54.50	126.0	475.08	27.30	5.34	157	475.
4:55	6:47	508.36	502.39	5.97	1190	1.72	2.65	4.90	3.38	0.37	0.18	505.28	53.00	122.2	471.32	33.96	6.64	148	471.
Arithmetical average value of "C"																		151.9	

Test No. 2 (explanatory notes relative to computations)

(3) (4) By rod readings down from reference elevation on hook gage pipe support.

(5) (6) (7) (8) Same as Test No. 1.

(9) (10) Flow in column (6) ÷ areas for 10″ and 12″ nominal diameters respectively.

(11) (12) $\frac{v^2}{2g}$ for velocities in column (9) (10) respectively.

(13) (27) Average elevation columns (3) and (4) minus 0.5 $\frac{v^2}{2g}$ for loss at entry.

(14) Average reading gage No. 4—160 lb. at one minute intervals.

(16) (20) Center line gage elevation 348.75 plus Column (15) plus Column (11).

(17) Column (13) minus Column (16).

(18) Column (17) ÷ 5,117 (equivalent 10″ for 5,490 ft. 12″ and 2,857 ft. 10″).

(19) (26) (33) By Williams-Hazen—Slide Rule.

(21) (28) Average reading gage No. 3—200 lb. at one minute intervals (4:47 to 4:59 and 6:46 to 7:00). Gage—50 lb. 5:00 t

(23) (30) Center line gage elevation 317.37 plus Column (22) plus Column (11).

(24) Column (20) minus Column (23).

(25) Column (24) ÷ 9,406.7 feet.

(31) Column (27) minus Column (30).

(32) Column (31) ÷ 14,523.7 feet (equivalent 10″ for 5,490 ft. 12″ and 12,263 ft. 10″).

TABLE 3

…n coefficient "C" in cast iron pipe lined with Bitumastic Enamel centrifugally applied

…roy Hills Township, New Jersery

…formula: … $= cr^{0.63} \times s^{0.54} \times 0.001^{-0.04}$

…l, 3-12″ … bend, 27-10″ × 6″ tees, 12-12″ × 6″ tees, 12-10″ gate valves, 5-12″ gate valves

…to station "C": Loss of head Sta. "A" to Sta. "C", Total	Loss of head Sta. "A" to Sta. "C", Slope "S"	Value of Williams-Hazen coefficient "C"	Data station "C" to station "D": Elevation of energy gradient (upstream) Sta. "C"	Elevation of gradient (downstream) Sta. "D": Gage reading	Elevation of gradient (downstream) Sta. "D": Equivalent head	Elevation of gradient (downstream) Sta. "D": Elevation of energy gradient	Loss of head Sta. "C" to Sta. "D": Total	Loss of head Sta. "C" to Sta. "D": Slope "S"	Value of Williams-Hazen coefficient "C"	Data station "A" to station "D": Elevation of energy gradient (upstream) Sta. "A"	Elevation of gradient (downstream) Sta. "D": Gage reading	Elevation of gradient (downstream) Sta. "D": Equivalent head	Elevation of gradient (downstream) Sta. "D": Elevation of energy gradient	Loss of head Sta. "C" to Sta. "D": Total	Loss of head Sta. "C" to Sta. "D": Slope "S"	Value of Williams-Hazen coefficient "C"
Feet	Ft./1000			Lb. per sq. in.	Ft. H_2O		Feet	Ft./1000			Lb. per sq. in.	Ft. H_2O		Feet	Ft./1000	
(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)
48.60	9.48	156	459.36	20.25	46.6	364.58	94.78	10.06	152	507.96	20.25	46.6	364.58	143.38	9.86	153
41.96	8.19	147	465.41	30.55	70.5	388.33	77.08	8.19	147	507.37	30.55	70.5	388.33	119.04	8.20	147
40.85	7.98	151	465.92	31.35	72.4	390.24	75.68	8.05	150	506.77	31.35	72.4	390.24	116.53	8.03	151
37.99	7.43	151	468.19	35.75	82.6	400.41	67 78	7.20	154	506.18	35.75	82.6	400.41	105.77	7.27	153
34.06	6.66	145	471.60	39.75	92.0	409.72	61.88	6.59	146	505.66	39.75	92.0	409.72	95.94	6.60	145
29.05	5.68	154	476.09	43.95	101.5	419.21	56.88	6.05	150	505.14	43.95	101.5	419.21	85.93	5.92	151
27.74	5.42	158	476.89	45.63	105.2	422.91	53.98	5.74	153	504.63	45.63	105.2	422.91	81.72	5.64	155
27.84	5.46	143	476.32	45.77	105.9	423.54	52.78	5.61	140	504.16	45.77	105.9	423.54	80.62	5.65	140
27.68	5.41	144	476.02	45.78	106.0	423.64	52.38	5.57	141	503.70	45.78	106.0	423.64	80.06	5.51	142
27.17	5.31	155	476.06	45.78	106.0	423.68	52.38	5.57	150	503.23	45.78	106.0	423.68	79.55	5.47	152
26.72	5.22	162	475.99	45.75	106.0	423.71	52.38	5.57	156	502.71	45.75	106.0	423.71	79.00	5.44	158
27.30	5.34	157	475.08	45.50	105.0	422.70	52.38	5.57	153	502.38	45.50	105.0	422.70	79 68	5.48	154
33.96	6.66	148	471.32	39.60	91.4	409.14	62.18	6.67	148	505.28	39.60	91.4	409.14	96.14	6.62	148
…………		151.9							149.3							150 0

… Gage … 50 lb. 5:00 to 6 45

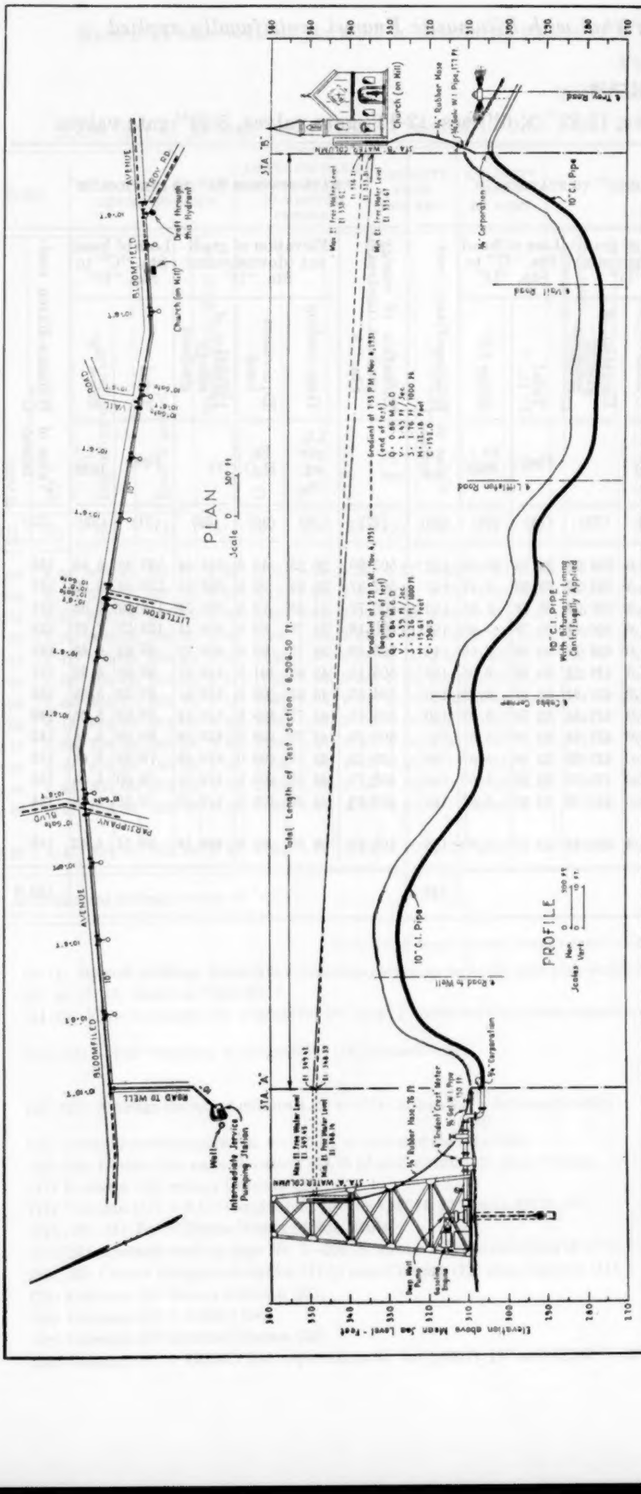

FIG. 5. CONTROLLING FEATURES OF TEST FOR DETERMINATION OF VALUE OF WILLIAMS-HAZEN COEFFICIENT "C" FOR CAST IRON PIPE LINED WITH CONTRIFUGALLY APPLIED BITUMASTIC LINING. UNDER ACTUAL FLOW CONDITIONS IN DISTRIBUTION SYSTEM, NOVEMBER 14, 1931

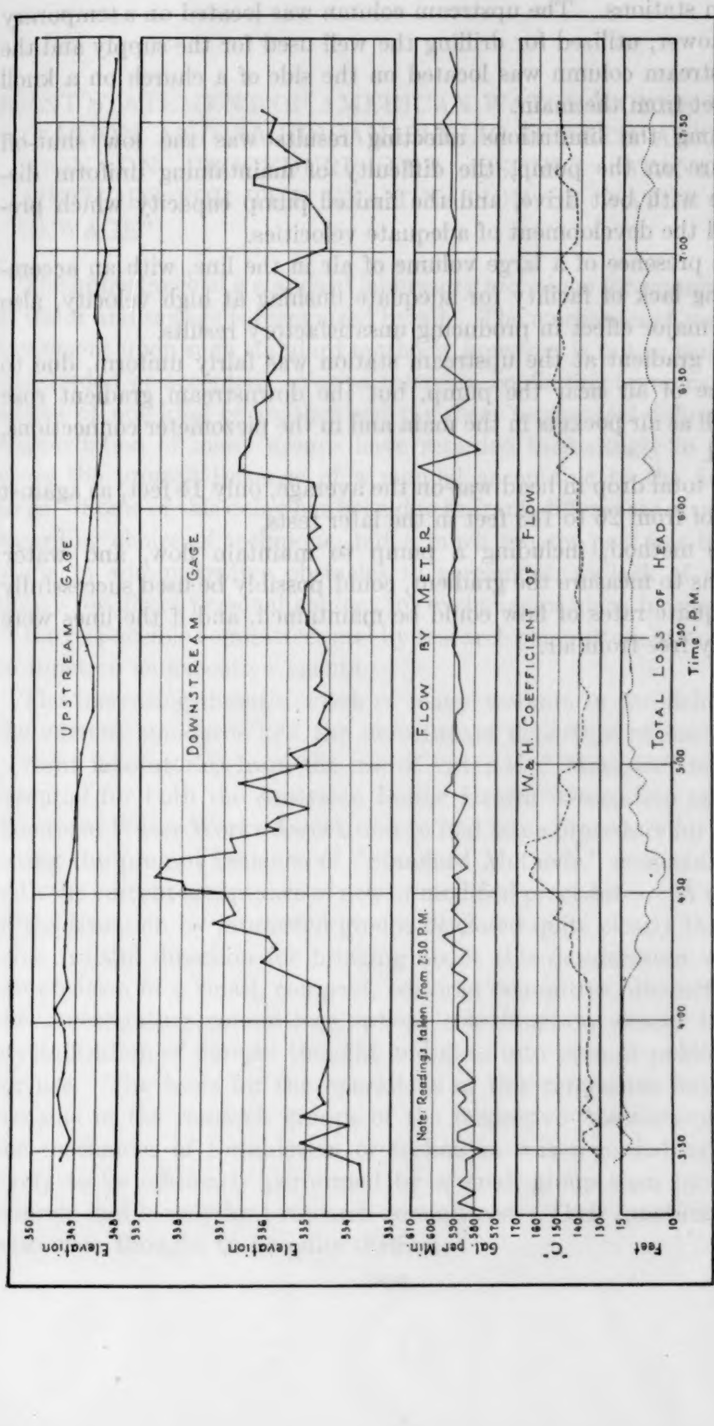

FIG. 6. RECORD OF OBSERVATIONS FOR TEST TO DETERMINE VALUES OF WILLIAMS-HAZEN COEFFICIENT IN CAST IRON PIPE LINED WITH CENTRIFUGALLY APPLIED BITUMASTIC LINING. TEST NO. A—NOVEMBER 4, 1931

stream stations. The upstream column was located on a temporary steel tower, utilized for drilling the well used for the supply and the downstream column was located on the side of a church on a knoll 150 feet from the main.

Among the limitations affecting results was the low shut-off pressure on the pump, the difficulty of maintaining uniform discharge with belt drive, and the limited pump capacity which prevented the development of adequate velocities.

The presence of a large volume of air in the line, with an accompanying lack of facility for adequate flushing at high velocity, also had a major effect in producing unsatisfactory results.

The gradient at the upstream station was fairly uniform, due to absence of air near the pump, but the downstream gradient rose and fell as air pockets in the main and in the piezometer connections, shifted.

The total drop in head was on the average, only 14 feet, as against losses of from 26 to 143 feet in the later tests.

This method, including a pump to maintain flow, and water columns to measure the gradient, could possibly be used successfully if adequate rates of flow could be maintained, and if the lines were entirely free from air.

JOINT STATEMENT OF AMERICAN WATER WORKS ASSOCIATION AND AMERICAN PUBLIC HEALTH ASSOCIATION ON PROCEDURE FOR ISSUING "STANDARD METHODS FOR THE EXAMINATION OF WATER AND SEWAGE"

The importance of a standard laboratory technique for examination of water and sewage is recognized by all. The mechanics of reaching agreement upon such standard methods, however, is not so generally understood or accepted. Varied interests, conflicting opinions, prejudices of one kind or another and the sheer mechanical difficulty of reconciliation of many groups have retarded increasingly in recent years the prompt issuance of a manual acceptable to the field at large. Some of this delay has been due to frank differences of opinion regarding choice of technique, but a much greater part has had its origin in administrative obstacles to agreement, to lack of opportunity for direct personal contact for exchange of ideas by members of the responsible committees and by research committees attempting to function as executive agents.

The increasing dissatisfaction of many workers in the field with the current standards and the unfortunate departure of many important laboratories from the use of "Standard Methods" made it essential for both the American Public Health Association and the American Water Works Association to find some procedure for facilitating the prompt issuance of "Standard Methods," commensurate with the current acceptance of new or modified procedures. A review of the situation by interested groups disclosed quite clearly that the most fruitful direction for bringing about this desideratum was in the creation of a small, compact, editorial committee, distinct from the investigating committees, whose sole function should be the crystallization of current thought and data into prompt publication for use. The basis for the operations of this committee naturally remains in the research groups of the respective Associations, but the mechanics of formulation of technique was regarded as more likely to be efficiently performed by a small group than by large, discrete and loosely knit research committees. Their functions likewise were thought to be quite distinct.

In the light of these conclusions, it appears advisable to inform the membership of the American Public Health Association and the American Water Works Association as to the changes which have been made in personnel for and methods of preparation of new editions of "Standard Methods for the Examination of Water and Sewage."

By an agreement which became effective in 1925, this manual of methods, which had formerly been prepared by the American Public Health Association alone, became the product of the cooperative activity of the two Associations. Since that time certain administrative reorganization has taken place or become effective in both Associations.

In the American Public Health Association, the Committee on Research and Standards has become the clearing house through which proposals for research projects are routed and to which the Association as a whole looks for coördination and direction of all research work. In the American Water Works Association the Committee on Water Works Practice occupies a similar position and performs a similar function.

While it is doubtless unnecessary to point out to most workers in this field, it may be worth while, for the benefit of a few who may not have thought the subject through, to recall that the establishment of such associational clearing houses of research and group activity is becoming more wide-spread and has its highest development in certain national organizations most conspicuous for their activity and excellent performance.

The Chairmen of the two Associations' Committees, after a review of the progress that was being made under the 1925 joint editorial agreement, have obtained the approval of their respective Boards of Directors to a reorganization of the groups engaged in editing "Standard Methods."

This reorganization consists briefly in the appointment of a joint Editorial Committee of six—three from each association—who have organized and have been functioning as a single committee since the Memphis Convention of the American Water Works Association.

The members of the Joint Editorial Committee are: John F. Norton, Detroit, Mich.; A. M. Buswell, Urbana, Ill.; L. M. Wachter, Albany, N. Y., representing American Public Health Association; Wellington Donaldson, New York; Paul Hansen, Chicago, Ill.; Chairman: Harry E. Jordan, Indianapolis, representing American Water Works Association.

This joint committee is empowered to carry on all the negotiations incidental to the preparation of new and revised editions from time to time, to the point of detailed and complete formulation of manuscript. After the completion of such manuscript, the material is to be submitted to the two parent Associations for the detailed ratification which those Associations may determine upon in their own practice. Funds have been allotted for the work of the committee.

In the American Water Works Association the manuscript will be submitted to the Chairman of the Committee on Water Works Practice who will, after satisfying himself and his committee, through whatever channels appear wise, as to the character of the material submitted, approve it and forward it to the Secretary of the Association. A like copy of the manuscript will be forwarded to the Chairman of the American Public Health Association Committee on Research and Standards. He will obtain the approval of the manuscript by his committee, after similar intra-Association review, and forward the material to the Secretary of the American Public Health Association.

The approved manuscript is then to be printed and bound under the direction of the General Secretary of the American Public Health Association. An annual accounting for expenses and receipts connected with the preparation and sale of Standard Methods will be made by the Secretaries of the two Associations.

It is planned that such research on methods as may be organized in the American Water Works Association shall take the form of appointments by the Chairman of the Water Purification Division with the advice of the Joint Editorial Committee and the approval of the Committee on Water Works Practice. Whenever it becomes evident to the Editorial Committee that a revision of an item of current procedure is needed or whenever the members of the Water Purification Division in regular meeting by their action indicate a similar conclusion, a research or revision group may be organized to study the particular subject. Such research groups are to be appointed for one year only and will be expected to make a report at a regular meeting of the Division. If, in the opinion of the Joint Editorial Committee, it is evident that the study has not reached a status that justifies a change in current procedure, the research group may be reappointed for another year with such additions to or changes in personnel as may be indicated. The policy of a definitely limited term of appointment of research groups is adopted as a means of accelerating results rather than of restraining the activities of any individual or group.

On the other hand, the Joint Editorial Committee is charged with the responsibility of contact with all research wherever organized as it may relate to the subject matter of Standard Methods. Whenever it becomes evident that the validity of research by any person or group is amply demonstrated, it is expected that a proper change will be made in the text of Standard Methods.

While the standing of the text on "Standard Methods for the Examination of water and Sewage" has rested in the past upon its adaptability to the determination of the sanitary and chemical characteristics of natural waters or those only slightly changed from their natural condition, it has become evident that the text should include data concerning determinations on highly modified waters for steam boiler purposes.

The Joint Editorial Committee is already in agreement with the Chairman of the A. W. W. A. Committee on Boiler Feed Water Studies, that whenever his committee is in agreement as to the technique of such determinations, the joint Editorial Committee will expand Standard Methods to contain the material.

The Joint Editorial Committee will endeavor to keep adequately informed as to the activities of water quality research committees in the National Electric Light Association, American Gas Association, American Railway Engineering Association, American Society for Testing Materials and other organizations of similar scope.

Its coöperation with the Committee of the American Chemical Society and the Society of American Bacteriologists is thoroughly established. Contact has already been established with the "Laboratory Methods Coördination Committee" of the Great Lakes and Ohio River Board of State Sanitary Engineers. It is anticipated that, whenever the Federation of Sewage Works Associations organizes research studies on methods applicable to the sewage field, the fullest harmony will exist and that whenever new or modified methods are developed, they will be readily accepted by the Joint Editorial Committee for inclusion in Standard Methods.

A new edition of Standard Methods will issue this autumn and it is anticipated that, with the coöperation of the various research committees, future revisions will appear as promptly as the extent of revised material justifies the expense of a new edition.

ABEL WOLMAN, *for the Committee on Research and Standards, A. P. H. A.*

MALCOLM PIRNIE, *for the Committee on Water Works Practice, A. W. W. A.*

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Rainfall and Stream Run-off in Southern California since 1769. H. B. LYNCH. Metropolitan Water District of Southern California, August, 1931. Comprehensive analysis of annual precipitation in Southern California, based on more or less accurate records going back to about 1815 and, prior to that time as far back as 1769, on written records of Spanish priests and others. Study shows that, in comparison with several periods of rainfall shortage which have occurred in past years, present rainfall deficiency to date can not be considered a major shortage. Comparison of rainfall and run-off from Southern California streams shows fluctuations from the normal run-off in general similar to those of the rainfall, but larger in relative percentage.—*David G. Thompson.*

On the Geology of the Source of the Water Supply to Ryojun (Port Arthur) City, S. Manchuria. K. TSURU, and S. MATSUSHITA. Ryojun Coll. Eng., Pub. no. 3 (repr. fr. J. Eng. Assoc. Manchuria 7: 37, 1930), 10 pp. (Engl. abs.: 1), 1930. Supply is drawn from ground water at Lungyenchuantun, 5 km. north. A system of gullies 5–6 meters deep and totalling 840 meters in length collects the water present in weathered fissures and solution channels. Hardness of water is 8° [145 p.p.m.?]; production of system is roughly 4,500 cubic meters [1.2 m.g.] per day.—*C. V. Theis; courtesy David G. Thompson.*

Air-Binding of Filter Beds. JOHN R. BAYLIS. Water Works and Sewerage, 77: 11, 379–384, November, 1930. Discussion of principles and laws affecting solubility and super-saturation of gases in water with some calculations as to amount of air that may be released upon filtration. Author attributes air-binding to air trapped in pore spaces before water is introduced. Brief bibliography is given.—*V. C. Fishel; courtesy David G. Thompson.*

Springs of Virginia. W. D. COLLINS, MARGARET D. FOSTER, FRANK REEVES, and R. P. MEACHAM. Commonwealth of Virginia, State Commission on

[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

Conservation and Development, Division of Water Resources and Power, Bulletin No. 1, 55 pp., 1930. Report on the discharge, temperature, and chemical character of springs in the southern part of the Great Valley.—*S. W. Lohman; courtesy David G. Thompson.*

The Underground Water of Aikawa, Chinchou, South Manchuria. TSUTOMU OGURA. Ryojun Coll. Eng., Pub. no. 2, (repr. fr. J. Eng. Assoc. Manchuria, 7 (36), 1930), 6 pp. (Engl. : 1), 1930. Ground water in the Aikawa area used for irrigation of rice fields, comes from two sources: (1) steeply dipping Middle Cambrian limestones from which the water issues as springs in an amount exceeding 800 tons per hour, and (2) shallow, spotted, gravelly beds in the alluvial materials covering most of the area. The deepest aquifer is from 24 to 30 meters below the surface.—*C. V. Theis; courtesy David G. Thompson.*

Lincoln's Water Supply Problem. G. E. CONDRA. Cons. Dept. of the Cons. and Survey Division, U. of Nebr., Bull. 4: pp. 12, 1930. Ground water used by Lincoln, Nebr., obtained mostly from the Dakota sandstone, is being pumped faster than it is being replenished by nature. Water is becoming saline. New sources of water for Lincoln are discussed, with reference to geological and topographical conditions.—*L. K. Wenzel; courtesy David G. Thompson.*

Some Interesting Tube Wells on the North Western Railway, India. J. VARDON. Punjab Engineering Congress, Paper No. 139, 1930, pp. 153–172g, 7 diagrams. Describes satisfactory results obtained in increasing supplies of water from wells along North Western Railway in India by using a method of gravel walling the wells patterned after methods used in United States. The gravel, however, was generally introduced by means of special wells drilled outside of the water well. Yield of one well increased from 1,000 to 6,500 gallons an hour by this method.—*D. G. Thompson.*

Notes on Rainfall and Evaporation. G. E. P. SMITH. Mon. Weather Review, 58: 6, 253–253, June, 1930. Under this heading, author gives short paragraphs in regard to several separate subjects as follows: differences in rainfall in certain parts of Arizona; transpiration from forests, as determined by observations on fluctuations of the water level in wells in mesquite and cottonwood forests in Arizona; "recapture of water supply" of Colorado River system when the present waste flow of Colorado River is used for irrigation and evaporated to reappear as rainfall; rainfall cycles; and summer rainfall of Arizona.—*David G. Thompson.*

Radio-activity of Stone Mountain Springs. JAMES A. HOOTMAN and W. S. NELMS. J. Am. Sci., [5] 21: 121, 37–38, January, 1931. Tests on radio-activity of water from springs at base of Stone Mountain, Georgia, are cited and authors conclude that most of the radio-activity is due to radium emanation originally dissolved in the water.—*A. N. Sayre; courtesy David G. Thompson.*

Der Temperaturverlauf im Sandboden (The Course of the Temperature in Sandy Soil.) R. SÜRING. Zeitschrift fuer Geophysik, 6: , 7, 285–291, 1930. Abstract in Bureau of Mines Information circular 6393, Geophysical abstracts, No. 18, p. 18, 1930. Gives information on diurnal course of temperature in upper strata and on yearly temperature distribution at greater depths.—*R. M. Leggette; courtesy David G. Thompson.*

Effect of Sealing on Acidity of Mine Drainage. R. D. LEITCH, W. P. YANT, and R. R. SAYERS. U. S. Bureau of Mines, Report of Investigation 2994, April, 1930. Gives acidity determinations of waters from open and closed sections of eight coal mines in Indiana. Conclude that sealing of abandoned sections of mines results in inhibiting acid formation.—*R. M. Leggette; courtesy David G. Thompson.*

Geology and Water Resources in Parts of the Peace River and Grand Prairie, Districts, Alberta. RALPH L. RUTHERFORD. Alberta Research Council, Geol. Survey Div., Rept. No. 21: 1–56, map, 1930. Gives results of geologic investigation in regard to possibilities of obtaining water supply for areas of unoccupied agricultural land. Surface water is most commonly used. Districts underlain by Wapiti formation are adequately supplied by artesian wells as deep as 500 feet, some of which flow. In most places, good supply of water can not be obtained from upper strata. In certain areas, deep wells are recommended to test artesian possibilities of the Dunvegan and the deeper-lying Peace River formations.—*S. W. Lohman; courtesy David G. Thompson.*

Moderne Waterwinnings-Inrichtingen met Toepassing van door Chemische Middelen Verkregen Bodemversteening (Modern Water-Supply Systems with the Application of a Chemical Process of Indurating Bottom Formations). A. LANG. H. de Jaargang 25: 249–252, December 12, 1930. Water glass and calcium chloride solutions are forced into underground beds of gravel and sand through small perforated tubes to make them impervious to water preparatory to driving tunnels through them. Proposed water-supply system of Düsseldorf, Germany, is described.—*A. N. Sayre; courtesy David G. Thompson.*

Das Untergrund-Wasser, seine Bildungweise und seine Erscheinungsformen (Ground water, Its Origin and Manifestation). F. RÖHRER. Das Gas- und Wasserfach, 12 pp., February 23, and March 2, 1929. Reprint. Author reviews briefly early attempts at rationalization of facts observed concerning ground water and discusses present theories of its origin and manifestation. Theories discussed are: infiltration theory, which attributes ground water to infiltration of rain water into the earth; condensation theory; and juvenile water theory. This discussion is followed by discussion of forms and methods of occurrence of underground water. Diagrams, tables, and graphs are used freely.—*A. N. Sayre; courtesy David G. Thompson.*

Zur Hydrogeologie der Quellen von Bad Dürkheim (The Hydrogeology of the Springs of Dürkheim, Bavaria). F. RÖHRER. Z. wissensch. Bäderkunde,

1930, H. 6 (separate, 14 pp.). Discusses chemical composition of the waters and geologic setting in the area of Dürkheim. Author concludes that only explanation possible for presence in spring and well water of large quantities of salt and of arsenic trioxide lies in a mixing of vadose and juvenile waters. The vadose water is derived by surface water passing downward and along Tertiary formations containing salt. The juvenile waters rise along a fault from Tertiary basalts, the nearest surface exposure of which is nine kilometers to the south. The juvenile water brings arsenic upward under pressure until it enters the Tertiary formation.—*A. N. Sayre; courtesy David G. Thompson.*

Ground Water Supplies in the Vicinity of Asbury Park. David G. Thompson. Bull. New Jersey Dept. Conserv. and Devel. **35:** 50 pp., 1930. In Asbury Park region, ground water is obtained from three horizons, the Mount Laurel-Wenonah, the Englishtown, and the Raritan. The consumption from the Raritan is greater than from either of the others. Pumping tests, observations on fluctuations of water level, and tests on sand samples show that water-bearing capacity of the Mount Laurel-Wenonah and Englishtown formations is low and that any large increase in consumption from either will probably result in inordinate loss of hydrostatic head Capacity of Raritan formation is much higher and it is recommended that future increases in consumption be supplied from this formation.—*David G. Thompson.*

Gas-Lift Method of Flowing Oil Wells (California Practice). H. C. Miller. Bull. U. S. Bur. Mines, **323:** 118 pp., 1930. Gives historical references and principles of gas-lift. Average efficiency of gas-lift installations in California is about 15 to 18 per cent. Gas-lift method not only increases production but increases ultimate recovery from reservoir sands. Discusses design, installation, operation, and cost of gas-lift plants.—*R. M. Leggette; courtesy David G. Thompson.*

Oil Pollution in the United States. Anon. Municipal Sanitation, **2:** 10, 507, October, 1931. Joint committee on oil pollution of American Engineering Council reports that oil pollution of coastal and inland waters of United States is declining. Committee is opposed to legislation imposing Federal jurisdiction over waters under state control. Where control has not been exercised, it has been due to laxity on part of state. One principal offender in this connection is United States Navy, which discharges oil from oil-burning warships. American Petroleum Institute reports that oil pollution from land planes is now largely the result of accident, or emergency, conditions.—*R. E. Noble.*

Sterilization of Water Mains. G. T. Luippold. Municipal Sanitation, **2:** 10, 493, October, 1931. Public expects and demands clean, safe water. Unless distribution system is free from contamination, water is not safe. Repairs and main extensions are frequent in every modern water works. Dirt and foreign material is certain to gain access to pipe line, particularly where laid in wet trenches. Should any of this material be contaminated, serious health menace would result. Chicago, New York, and Baltimore sterilize

every section of new main before placing same into service. *Methods of Sterilizing Water Mains.* Hypochlorite of lime has proved unsatisfactory, because it loses strength very rapidly when exposed to air. Liquid chlorine is now almost universally used, either by direct dry-feed method in which dry gas is introduced through diffuser, or by solution-feed method, which is better because it gives much better diffusion. Small new systems install solution-feed type of chlorinator in main pumping station introducing chlorine on suction side of pump, and passing it into entire system in sufficient quantities to give residual at extreme ends of system. Larger systems divide the piping up into sections by means of valves, sterilizing one section at a time. For extensions, chlorine solution is applied in new main at connection near shut-off valve separating new and old sections. On each side of shut-off valve, pressure gauge is tapped into line and valve is then opened enough to bring pressure differential between the two sides to between 3 to 1 and 5 to 1. Differential is necessary because chlorine injector requires for its operation pressure approximately three times greater than that against which chlorine solution is introduced. Blow-off points at various hydrants and at ends of new line are left open and new main is flushed with velocity that will scour out dirt. Chlorine is then introduced. After dirt has been flushed out and chlorine residual has appeared at all blow-off points, valve is closed and chlorine-laden water left in main for an hour or longer. Main is then flushed with sterilized water until free from residual chlorine, or until new main shows same residual as obtains in old main. It may be necessary to repeat sterilization. Main should be kept out of service until bacteriological findings are negative. Sound investment for any municipality is a small truck upon which is mounted gas-engine-driven pump with solution-feed type of chlorinator and cylinder of chlorine. *Amount of Chlorine Required* is dependent upon character of ground through which main has been laid and consequent amount of infected material to be sterilized. Initial dosage introduced, as indicated, at control point will ordinarily run to from 0.5 p.p.m. to 10 or 15 p.p.m., while residual present at most remote point will run to from 0.1 to 1 p.p.m., depending upon conditions. Value of sterilizing new mains cannot be overestimated. Not only should newly-laid mains be sterilized, but lines which have failed should be sterilized throughout before again being placed in service.—*R. E. Noble.*

Water Purification. A Retrospect. ALEXANDER HOUSTON. Municipal Sanitation, 3: 4, 148, April, 1932. *Chlorination.* The dawn of chlorination was at Lincoln, England, in 1905. Now most of London's water supply is chlorinated before, or after, filtration. Chief ways of *avoiding taste troubles* are: (1) *Super-chlorination,* or destroying potentially taste-imparting substances by application of major dose of chlorine. Remaining chlorinous taste is removed by sulphur dioxide, sulphite, or activated carbon. Usual chlorination dose is approximately 0.25 p.p.m.; super-chlorination dose may be 1.0 to 2.0 p.p.m. Super- and de-chlorination provide a wide margin of safety against pollution. (2) *Permanganate and Chlorine.* Permanganate prevents and removes taste and may be added before, with, or after, chlorination. Usual doses are 0.2 to 0.8 p.p.m. Disadvantage is brownish precipitate

produced. Permanganate in doses of 0.25 to 0.5 p.p.m. completely removes aromatic geranium-like taste due to *Tabellaria*. Permanganate may be practically useless when growths are decomposing and taste results from putrefactive processes. Here use (3) *Activated Carbon.* (4) *Ammonia and Chlorine.* (*a*) Bring ammonia and chlorine together in minor volume of water to produce monochloramine, or dichloramine, compound, and add this to the water in bulk. Proportion of ammonia to chlorine is from 1.0:1.4 to 1.0:8.0. Or (*b*), add first ammonia (from 0.1 to 0.2 p.p.m., in terms of nitrogen) to whole body of water to be treated and then chlorine (0.25 p.p.m.). Either method is most successful for sterilization and for avoiding taste troubles. During years of treatment of river water in aqueduct at Staines, *chlorination has saved* £120,000, difference between cost of coal for pumping and cost of chemicals for sterilization. *Vitality of the Typhoid Bacillus in River Water.* Concluded that four weeks storage is sufficient; that "uncultivated" bacilli (i.e., bacilli which had never been grown outside of animal body) die more speedily than "cultivated;" and that typhoid bacilli survive longer at lower temperatures. *Search for Typhoid and Paratyphoid Bacilli in River Water and Sewage.* Experimentation was held to show that pathogenic bacteria cannot be uniformly present in river water, or even in sewage, unless in very small numbers. Recently this view has had to be modified considerably in case of sewage of towns which have experienced epidemic of paratyphoid. In one such case, sewage contained, months after epidemic, hundreds of paratyphoid bacilli. Editorial on epidemics at Hanover and Montreal by Wolman (A. W. W. A. 1927, **18**: 4) is given. *Vitality of the Cholera Vibrio in River Water.* After one week's storage, cholera vibrio is reduced 99 per cent. Article from Fifteenth Annual Report (1921) on detection of cholera vibrio in water is quoted. *Storage.* Author found that raw River Lee water, after storage in Walthamstow Reservoirs, was greatly improved, bacteriologically. If prefiltration waters can be pronounced epidemiologically "safe," the perfect working of the filtration process becomes secondary, aside from producing physically attractive product. *Algal and Other Growths.* Copper sulphate is not recommended as algicidal agent (Fifteenth Annual Report, p. 52). Better for this purpose is either preliminary rapid filtration, or excess lime treatment. *Rapid Filtration.* Thirteenth Research Report (1920) showed that primary filters working at very high rates (100, or more, gallons per square foot per hour) could remove most algal growths from stored water. This led to adoption of double filtration process, resulting in great saving of capital and of working costs. *Resistance to Filtration.* Twelfth Research Report (1916) makes clear that, in practice, many potentially taste-producing growths fail to give rise to taste troubles. Reference is made to the Twelfth and Thirteenth Research Reports, and to the Eleventh, Twelfth, Thirteenth, and later Annual Reports, touching upon microscopical growths. *Excess lime* acts as clarifying, softening, and purification (including precipitation of algal and other growths) agent and also sterilizes. In ordinary lime-softening process, lime dosage falls just short of completely removing temporary hardness. In excess lime method, overdose applied suffices to render water absolutely safe bacteriologically. Bactericidal dose is about from 10 to 20 p.p.m. of CaO. Excess lime may be removed by carbon dioxide. *Leather*

Bacillus. Coliform organisms were invariably present in water from certain tap, despite fact that output from filters throughout well were satisfactory. Trouble was due to growth of organism now called "leather bacillus" in leather washer of tap itself. This microbe was found to be capable of multiplying almost indefinitely on "susceptible" washers (reference, p. 9, Twelfth Research Report). *Gulls,* if not a menace, are a great nuisance, frequenting Board's reservoirs and filter beds in almost incredible numbers. Their droppings contain at least one million *B. coli* per gram, indistinguishable from those of human origin. Wires stretched at intervals over body of water frighten gulls away. *Bacteriophages.* Short discussion given. *Dissolved Oxygen.* Lists reports on condition of River Thames as to dissolved oxygen. *Sterilization of new mains.* Ordinarily, chlorination treatment kills *saprophytic leptospira. Accidents.* Only safe procedure is to forestall accidents by removing, or preventing, all occasions thereof. *Conclusions.* (1) Choose virgin source of water supply. (2) Do not economize in water purification. (3) Do not allow water to become contaminated in distribution system. (4) Safe water supply is insurance policy of tremendous value. (5) Specifically contaminated water supply strikes disastrously a large proportion of consumers. (6) A water-borne epidemic and its aftermath spell ruin to a community. (7) Encourage water research. (8) Water should be safe to drink and pleasant to taste and to look at. (9) Impure water can be banished forever.—*R. E. Noble.*

The Influence of Higher Temperatures and Salt Additions on the Lime-Carbon Dioxide Equilibrium in Water and the Formation of the Rust-Resistant Coating. J. Tillmans, Paul Hirsch and Wilh. R. Heckmann. Gas- u. Wasserfach, **74**: 1–9, 1931. From Chem. Abst., **25**: 1310, March 20, 1931. Previous articles (cf. C. A. **23**: 1973, 4987) have indicated that for given concentration of free carbon dioxide in water at given temperature there is corresponding concentration of bicarbonate ion (HCO_3^-) below which calcium carbonate will not be precipitated and is, therefore, unable to form protective coating. Previous data on this equilibrium have been available only for 17°; this paper presents experimental data for 40°, 60°, 80°, and 100°. Free carbon dioxide was determined by method of Tillmans and Heublein (C. A. **6**: 3301) and bicarbonate, by titration against 0.1 normal hydrochloric acid, with methyl orange. These values were plotted for above temperatures as p.p.m. carbon dioxide. Any concentration of combined carbon dioxide greater than that indicated by equilibrium curve for given temperature will result in precipitation of calcium carbonate and formation, if oxygen is present, of rust-protective coating of iron oxide and calcium carbonate. Water may be "lime aggressive" at atmospheric temperature, forming no protective coating, but non-aggressive at higher temperatures, forming protective coating. Several practical examples are given. Low concentrations of sodium chloride have only slight effect on lime-carbon dioxide equilibrium, but magnesium chloride concentrations above 20 p.p.m. (as MgO) require much greater bicarbonate concentration (with given free carbon dioxide concentration) to form protective coating—*R. E. Thompson.*

The Drinking Water and Sewerage Project of Linares, Nuevo León. ALEJANDRO SAVA. Ingenieria, 4: 349–57, 1930. From Chem. Abst., 25: 1304, March 20, 1931. Complete description of project.—*R. E. Thompson.*

The East Hamburg, Germany, Water Supply. W. HOLTHUSEN. Gas- u. Wasserfach, 73: 1180–3, 1930. From Chem. Abst., 25: 1304, March 20, 1931. Discussion of plants to supply Hamburg, and territory east of it, with water.—*R. E. Thompson.*

Corrosion and Discoloration in the Water Supplies of Perth, W. A. H. E. HILL. Chem. Eng. Mining Rev., 23: 35–40, 1930. From Chem. Abst., 25: 1310, March 20, 1931. Water Supply of Perth comprises three distinct types of water: (1) shallow catchment, or hills water, (2) shallow bore, or artesian, water from depths between 550 and 800 feet, and (3) deep bore water, from depths between 1000 and 2100 feet. Water discoloration, in certain well-defined areas, varies between slight brown tinge and deep coffee-brown, and is due to rust. Corrosion problem involves deterioration of, and reduction in carrying capacity of, city mains and accumulation in them of rust deposits, with resultant "dirty" water. Summarized results follow. Discoloration is due to extremely light flocculent type of rust formed in mains, particularly by hills catchment water. Lime treatment of water and improved circulation in pipes alleviate trouble considerably. BAYLIS method of establishing calcium carbonate protective coating on inside of mains by lime treatment to pH above solubility curve for calcium carbonate failed because of high concentration of chlorides.—*R. T. Thompson.*

New Rust-Preventive Pigments as Substitute for Red Lead. JUL. F. SACHER. Chem.-Ztg., 54: 781–2, 1930. From Chem. Abst., 25: 1397, March 20, 1931. Pigments ordinarily used as rust preventives are listed in order of their rust-resisting qualities. Red lead possesses disadvantages of being poisonous and of being difficult to cover with other paints. New lead pigment has appeared, under trade name Arcanol, lead in which exists partly in metallic state and partly as oxide. Paints made from this material dry in 8 hours at 0°, possess high covering power and great expansibility and are very resistant to action of hot water or steam. New oil-free rust-preventive paint suitable for use in tropics dries exclusively by solvent evaporation. It is impermeable to air, water, or vapor, and resistant to sea water, hot vapor, flue gas, or acid vapor, stable toward cold and unaffected by temperatures of from 200° to 250°. Numerous uses are suggested for this material.—*R. E. Thompson.*

Hard Industrial Water in the Textile Industry. M. MÜNCH. Z. ges. Textil-Ind., 33: 664–6, 678–9, 1930. From Chem. Abst., 25: 1387, March 20, 1931. Description of harmful action of lime soaps on textiles and of method of prevention.—*R. E. Thompson.*

The Radioactive Properties of Rocks, Soils, Crude Oil, and Waters from Southern California. J. LLOYD BOHN. J. Franklin Inst., 210: 461–72, 1930; cf. C. A., 24: 4699. From Chem. Abst., 25: 1437, April 10, 1931. Measure-

ments are given on waters from Lake Arrowhead, Arrowhead Hot Springs, Harlem Hot Springs, and Pacific Ocean, and from numerous wells and tunnels of Pasadena and from neighboring water supplies.—*R. E. Thompson.*

Colorimetric Determination of the Sulfate Ion in Water, Coal, etc. P. GUARNIER. Ind. ital. cons. alin., **3**: 161, 1930; Chimie & industrie, **24**: 814. From Chem. Abst., **25**: 1458, April 10, 1931. Method consists of treating 100 cc. of water at boiling temperature with hydrochloric acid solution of barium chromate, neutralizing with calcium carbonate, removing precipitate by filtration, and comparing color of filtrate with that of standards.—*R. E. Thompson.*

Building up Metals with Electric Arc Welding. K. TEWES. Giesserei-Ztg., **27**: 585–9, 1930. From Chem. Abst., **25**: 1478, April 10, 1931. In arc welding, it is customary to add to the electrodes materials which will improve quality of weld. Tin and manganese have been widely used to bring about deoxidation of weld, but titanium appears to be more suitable. It combines energetically with oxygen and materially reduces quantity of oxides in weld. Titanium oxide formed readily floats to top and can be removed with slag. Greater homogeneity of weld results in improved mechanical properties. Titanium also tends to form nitrides, which greatly increase hardness of steel.—*R. E. Thompson.*

Soft Steel Welds Deposited by the Electric Arc. D. ROSENTHAL and M. MATHIEU. Compt. rend., **191**: 484–6, 1930. From Chem. Abst., **25**: 1478, April 10, 1931. Nature of weld of soft steel bars produced by electric arc depends on manner in which molten metal is protected from oxidation. Metallographic examination shows that welds protected with covered electrode have more regularly formed grains than non-protected welds. X-ray diagrams show existence of strains in non-protected welds.—*R. E. Thompson.*

Influence of a Small Addition of Copper on the Corrosion Resistance of Structural Steel. O. BAUER, O. VOGEL and C. HOLTHAUS. Mitt. der deut. Materialprüfungsanstalt, Sonderheft **XI** 25 pp., 1930. Extended abstract in Metals and Alloys, **1**: 890–5, 1930. From Chem. Abst., **25**: 1479, April 10, 1931. Samples covered range of soft steels, structural steels, and 0.25 to 0.35 per cent carbon steels. Tests were conducted in various solutions, including distilled water, river water, humic acid solution, etc. Data and graphs are given.—*R. E. Thompson.*

Some Properties of Protective Films on Metals. ERNEST S. HEDGES. Chemistry and Industry, **50**: 21–5, 1931. From Chem. Abst., **25**: 1478, April 10, 1931. Review of film properties is given, with special consideration of work carried out by EVANS and by author. Theory of anodic polarization is offered.—*R. E. Thompson.*

The Mechanism of the Suppression of Corrosion Velocity by Colloids. W. BECK and F. v. HESSERT. Z. Elektrochem., **37**: 11–20, 1931. From Chem. Abst., **25**: 1478, April 10, 1931. Decrease in corrosion velocity of iron in

water, or in strong acid solutions, is due to presence of hydrophilic colloids, and is dependent on type as well as on concentration of added colloid. Authors suggest possibility of formation of coagulation layers upon surface of metal, due to action of iron ions upon negatively charged particles of hydrophilic colloid. Effects of various colloids were studied.—*R. E. Thompson.*

Protection of Large Steel Structures from Rust. HENRY E. WEITKAMP. Het Gas., **50**: 480–4, 1930. From Chem. Abst., **25**: 1478, April 10, 1931. A review.—*R. E. Thompson.*

Vital Conditions of Ferruginous Bacteria. (Mlle.) I. TUROWSKA. Bull. intern. acad. polonaise, 1929B I: 255–82. From Chem. Abst., **25**: 1548, April 10, 1931. Iron bacteria are found in certain wells and other waters of Poland in which iron content is usually greater than 2 p.p.m. and reaction, between pH 5.88 and 7.60. Large concentrations of salts appear to be unfavorable. The bacteria are able to withstand temperatures down to 0° —*R. E. Thompson.*

Determination of Silicic Acid in Water. W. STEFFENS. Chem.-Ztg., **54**: 996–7, 1930. From Chem. Abst., **25**: 1608, April 10, 1931. Colorimetric method depending on formation of yellow silicomolybdate is described again, preference being given to potassium chromate solutions, rather than picric acid solutions, as color standards.—*R. E. Thompson.*

NEW BOOK

Water Purification Control. EDWARD S. HOPKINS. Williams & Wilkins, 1932, 131 pages, price $1.75. The subject of water purification is most interesting. It leads into the realms of chemistry and engineering, bacteriology, biology, physics and public health. The ideal operator in charge of purification has a knowledge of all these and in addition is a mechanic, executive and diplomat. The author does not discuss the last-named personal qualifications, but he has covered the rest of the field in 122 pages, a feat requiring considerable literary art, as well as knowledge of the subject. Obviously this could be done best by one in daily contact with an important plant which is among the foremost in the application of scientific method. The author qualifies in this respect and shows skill in his simple, concise statements of good practice. The chapter on Filtration is especially timely and authoritative in the discussion of filter washing and maintenance. The chapter on coagulation includes very good discussions of pH and other laboratory control methods. The chapters on Disinfection and Taste and Odor Control are well handled also. These subjects are amply treated in current literature, but the average operator will get a sounder view of the value of various methods from the practical surveys of these fields which the author gives. It is to be regretted that the control of water softening could not be included. But in spite of the fact that the book is written from the standpoint of the soft water east of the Appalachians, it is easily the best elementary text available for the operators of plants in any part of the United States, and one that should be in every plant library.—*Charles H. Spaulding.*